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In this issue: A study of vehicle use of traffic lanes and shoulders on two-lane primary rural highways.

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C. M. Billingsley, *Editor*

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IN THIS ISSUE

U. S. DEPARTMENT OF COMMERCE
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BUREAU OF PUBLIC ROADS
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CHARLES D. CURTISS, Commissioner

Driver Behavior Related to Types and Widths of Shoulders on Two-Lane Highways

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported [1] by ASRIEL TARAGIN, Head, Driver Behavior Section

In this article the results of a study of vehicle placements and speeds observed on two-lane primary rural highways are discussed. Speed and placement data were recorded for 89,000 vehicles at 66 locations in nine Western States.

This analysis is concerned primarily with bituminous pavements, although a limited number of sections paved with portland cement concrete were included in the study.

Lateral position and speed studies were conducted on test sections with various types and widths of shoulders—gravel shoulders, combinations of bituminous materials and gravel, and bituminous-paved shoulders. On some sections of bituminous pavements and shoulders, there was sufficient contrast in the appearance of the traffic lanes and shoulders for drivers to differentiate between them; on other sections, the bituminous-paved traffic lanes and shoulders appeared to be a continuously paved roadway.

Commercial vehicles made much greater use of the shoulders than passenger cars. On test sections with matching bituminous-paved traffic lanes and shoulders, nearly 80 percent of the commercial vehicles meeting other vehicles traveled partly on the shoulder. The use of a 2-inch solid white stripe to separate the traffic lane from the shoulder had the effect of reducing shoulder encroachment by about 50 percent.

Shoulder surface types of comparable widths did not seem to influence vehicle speeds. Passenger-car speeds for all locations studied averaged 55 miles per hour and commercial vehicles, 48 miles per hour.

APPROXIMATELY 94 percent of the existing mileage of primary rural roads in the United States were designed to accommodate two lanes of traffic. In the Western States, the greatest proportion of these roads are surfaced with bituminous materials. The normal practice in the past was to construct gravel shoulders adjacent to the traveled lanes, but in recent years, a number of the Western States have adopted the practice of also paving the shoulders with bituminous materials.

In most cases there has been a definite distinction between the appearance of the traveled lanes and the shoulders, either in color or texture or both. On a small but significant mileage of these rural roads, however, it is impossible to distinguish between traffic lanes and shoulders. These roads appear to be two-lane highways with 20-foot lanes and no shoulders instead of the normal 12-foot lanes with 8-foot shoulders. Constructing highways in this manner has occasioned much concern among engineers not only because shoulder areas are not usually designed to carry the loads of the traffic lanes, but drivers tend to operate on these sections as if they were four-lane undivided highways.

This concern resulted in a study of driver behavior and the effect of contrasting traffic lanes and shoulders on safety of operation as compared with construction practices where shoulders cannot be distinguished from traffic lanes. The study was a series of cooperative undertakings between the Bureau of Public Roads and the State highway departments of Arizona, California, Colorado, Idaho, New Mexico, Oregon, Texas, Utah, and Washington. Field work was started in March 1955 and continued for 6 months. Vehicle speeds and placements were recorded for each of the several sections selected for study, and passing maneuvers were observed over a one-half mile length of highway at a limited number of locations. Observers noted the types of vehicles involved in each passing maneuver and their approximate transverse positions.

Summary of Major Findings

The results of this study point to a number of significant aspects of driver behavior with respect to the use of shoulders on rural two-lane highways carrying light to moderate traffic volumes. Included in the study were road sections with gravel shoulders adjacent to paved traffic lanes, bituminous-paved shoulders contrasting in appearance with the paved traffic lanes, and bituminous-paved shoulders matching the paved traffic lanes. Several types of edge striping were investigated to determine their effect on the lateral positions of vehicles traveling on the test sections.

The more important findings of this study follow:

1. Although vehicle speeds were not affected by shoulder surface types, a relation did exist between speeds and lateral positions of vehicles on test sections with matching bituminous-paved traffic lanes and shoulders. The average position of the slower moving vehicles, whether passenger cars or trucks, was closer to the shoulder of the highway than that of the faster moving vehicles.

2. Commercial vehicles encroached on shoulders to a greater extent than passenger cars. This was particularly apparent on bituminous-paved sections with matching traffic lanes and shoulders where nearly 80 percent of the trucks meeting other vehicles traveled partly on the shoulder. On sections with contrasting traffic lanes and shoulders, encroachment on shoulders was one-third to one-half that of sections with matching traffic lanes and shoulders.

3. Average clearances between bodies of meeting vehicles were about 6 feet on test sections with wide gravel shoulders, about 7½ feet on sections with a combination of bituminous and gravel shoulders, and over 10 feet on bituminous-paved sections with matching traffic lanes and shoulders.

4. Nearly 30 percent more passing maneuvers were observed on test sections with matching traffic lanes and shoulders in comparison with all other combinations of contrasting lanes and shoulders.

5. In the study of lateral positions of vehicles, it was evident that the greater the degree of contrast in appearance between traffic lanes and shoulders the greater the tendency for drivers to operate within the confines of the traffic lanes. This concentra-

[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

Table 1.—Number of two-lane highway test sections included in study, classified according to surface types and widths of traffic lanes and shoulders

Group	Number of locations	Number of vehicles	Traffic lanes		Shoulder width		Appearance of bituminous shoulder compared with traffic lanes
			Width	Type	Bituminous	Gravel	
			Feet		*Feet*	*Feet*	
1	3	4,000	12	Bituminous		3–5	
2	8	8,900	12	do		6–10	
3	6	9,900	12	do	4	4–6	Contrasting.
4	15	22,800	12	do	6–10		Do.
5	16	14,600	12	do	8		No contrast.
6	3	4,900	12	Portland cement concrete	8–10		
7	3	5,100	12	do		9	
8	4	7,900	11	Bituminous	6–8		Contrasting.
9	3	2,400	12	do	8		No contrast.[1]
10	2	2,600	12	do	8		No contrast.[2]
11	3	5,900	12	do	4	6	Contrasting.[2]

[1] Stripe near outside edge of shoulder.
[2] Stripe near outside edge of traffic lane.

Table 2.—Average speeds of vehicles traveling on two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and shoulders of various surface types and widths

Vehicle classification	Average speeds for two-lane highways with—				
	Gravel shoulders, 3–5 feet	Gravel shoulders, 6–10 feet	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders, 8 feet [3]
Passenger cars:	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Free moving	53.4	58.2	55.9	56.3	54.3
Meeting other vehicles	53.2	57.2	55.0	56.0	53.3
All other passenger cars	52.5	56.6	54.0	55.4	53.2
Commercial vehicles:					
Free moving	46.2	49.3	47.2	48.4	47.5
Meeting other vehicles	45.5	50.1	48.7	48.2	49.1
All other commercial vehicles	46.7	48.6	46.6	48.1	46.7

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

tion of lateral placement was particularly noticeable on bituminous and concrete test sections with gravel or grass shoulders.

6. Shoulder edge stripes closer than 1½ feet from the outside edge of bituminous-paved shoulders (18½ feet from the centerline of the roadway) had no effect on vehicle speeds or lateral positions.

7. A 2-inch solid white stripe painted 8 feet from the outside edge of the shoulder or 12 feet from the centerline of the roadway having matching bituminous-paved lanes and shoulders had the effect of reducing shoulder encroachment by about 50 percent as compared to test sections without edge stripes.

8. On bituminous pavements with combination-type shoulders of 4 feet of bituminous material and 6 feet of gravel, a 4-inch solid yellow stripe placed 13 feet from the centerline of the roadway was very effective in reducing shoulder use, especially by trucks.

9. On two-lane, 22-foot bituminous pavements with contrasting 6- to 8-foot bituminous shoulders the lateral positions of vehicles and clearances between meeting vehicles were about the same as for two-lane, 24-foot bituminous pavements with 6- to 10-foot gravel shoulders.

Locations Studied

Only level tangent sections having a cross-section design typical of that used in the particular State were included in the program. The number of locations studied in each of the several groups of roadway sections is shown in table 1. Speed and placement data were obtained for 89,000 vehicles at 66 locations during the daylight hours; data were also recorded at night at the locations included in groups 9–11.

The average speed and placement data for the several groups of locations are representative of driver behavior for the cross-section designs shown. Even though the speeds and placements at some locations with a given cross section were somewhat different from those at other locations having the same cross section, the variation was remarkably small.

The information recorded for groups 1–5 is of primary interest insofar as bituminous surfaces are concerned. These five groups represent five different shoulder widths and surface-type combinations on two-lane highways having 12-foot bituminous-surfaced lanes. A more detailed description of the types of shoulders included in these groups follows:

Group 1.—Gravel shoulders, 3 to 5 feet wide. The material consisted of gravel or loose stone chips having the same appearance as gravel. The cross slope of the shoulder was considerably greater than the traffic lanes.

Group 2.—Gravel shoulders, 6 to 10 feet wide. The cross sections, other than the width of the shoulder, were approximately the same as group 1. Included also were shoulders treated with oil and covered with crushed gravel.

Group 3.—Four-foot bituminous-paved shoulders contrasting in appearance with the traffic lanes. Gravel shoulders, 4 to 6 feet wide, extended beyond the paved shoulder.

Group 4.—Bituminous-paved shoulders, 6 to 10 feet wide, contrasting with the traffic lanes. Beyond the paved shoulder, the roadbed sloped sharply to the ditch line. On several sections the slope of the shoulder was noticeably greater than that of the traffic lane.

Group 5.—Bituminous-paved shoulders, 8 feet wide, having the same appearance, texture, and riding qualities as the traffic lane. To all outward appearance these roads were two-lane highways with 20-foot lanes and no shoulders.

Groups 6–11.—These groups include cross sections of varied shoulder widths and surface types, some of which are typical in certain States. Special studies of edge striping and signs were conducted on sections included in these groups.

Table 3.—Percentages of vehicles traveling below 40 and above 60 miles per hour on two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and shoulders of various surface types and widths

Vehicle classification	Percentage of vehicles traveling at indicated speeds on two-lane highways with—				
	Gravel shoulders, 3–5 feet	Gravel shoulders, 6–10 feet	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders, 8 feet [3]
VEHICLES TRAVELING BELOW 40 MILES PER HOUR					
Passenger cars:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Free moving	10.4	6.4	6.6	6.7	8.6
Meeting other vehicles	11.6	5.8	7.7	7.2	11.6
All other passenger cars	14.2	7.2	8.9	7.5	9.6
Commercial vehicles:					
Free moving	25.9	14.9	19.3	14.8	20.8
Meeting other vehicles	29.1	10.5	19.1	18.4	16.9
All other commercial vehicles	25.0	13.2	18.6	17.0	24.4
VEHICLES TRAVELING ABOVE 60 MILES PER HOUR					
Passenger cars:					
Free moving	29.6	45.7	39.3	42.0	31.6
Meeting other vehicles	27.9	41.3	34.9	40.9	26.3
All other passenger cars	29.0	39.0	31.2	38.6	28.9
Commercial vehicles:					
Free moving	9.0	12.3	9.3	10.5	11.0
Meeting other vehicles	3.5	15.4	13.5	10.9	12.8
All other commercial vehicles	9.7	11.1	7.3	10.2	9.9

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

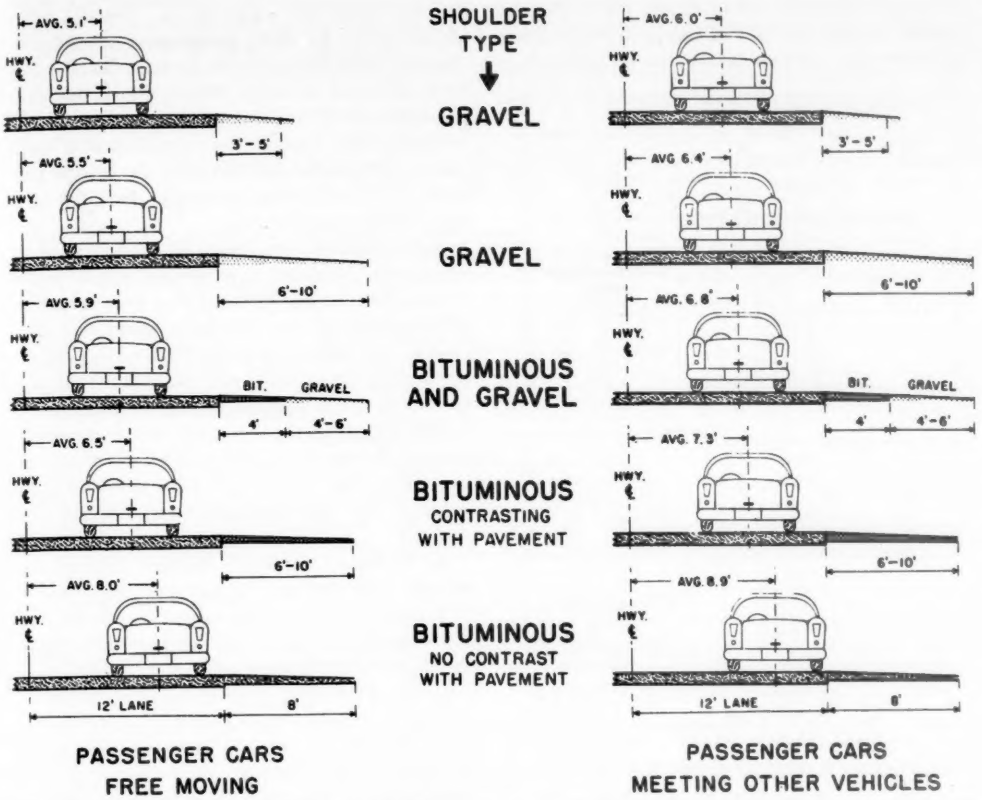

Figure 1.—Average lateral positions of passenger cars on two-lane bituminous-paved rural highways.

Classification of Data

The data collected consisted primarily of speeds and placements of all vehicles as they passed a selected point of observation. Each vehicle was classified as to whether it was free moving, meeting another vehicle traveling in the opposite direction, trailing a vehicle in the same direction, passing another vehicle, or being passed. A vehicle not belonging in any of the foregoing groups was classified as "other." It has been found that the best measure of the effect on driver behavior of the variables under study is provided by the free-moving and meeting vehicles. Reliable comparisons of speeds and placements were possible between vehicles in these two groups. The remaining groups, with the exception of vehicles passing other vehicles traveling in the same direction, were combined into one group and identified as "all other vehicles." The number of vehicles involved in passing maneuvers during the periods of study were too few to permit an analysis which would produce significant results. Because of their position in the left traffic lane (negative placement), the passing vehicles were not included in any of the groups.

Comparative data are therefore shown for vehicles in the following three groups:

Free-moving vehicles.—Those vehicles which were, for practical purposes, uninfluenced by other traffic on the highway when speeds and transverse positions were recorded. About 55 percent of the vehicles studied were in this group.

Meeting vehicles.—Those vehicles that might have been directly affected by opposing traffic but not by vehicles traveling in the same direction. Clearances between the bodies of these vehicles were calculated from the placement data. About 15 percent of the vehicles were in this group.

All other vehicles.—Those vehicles which were not classified as free moving or meeting other vehicles. These vehicles, which constituted 30 percent of all traffic, probably included some that were influenced in speeds and placements by other traffic.

Speeds and placements were analyzed separately for passenger cars and commercial vehicles at each location studied. Data obtained at the several sites having identical highway cross-section geometry were then combined.

Vehicle Speeds

Table 2 shows the average speeds of vehicles recorded at 48 locations included in groups 1–5. There is no great variation between the speeds on sections with shoulders of approximately the same width. A comparison of speeds observed on sections with gravel shoulders, averaging 4 and 8 feet wide, indicates that the various classes of vehicles traveled about 2 to 5 miles per hour faster on sections with the wider shoulders. Passenger-car speeds on all sections averaged 55 miles per hour and trucks averaged 48 miles per hour.

Table 3 provides a comparison of the percentages of vehicles traveling below 40 miles per hour and exceeding 60 miles per hour. Again, it is evident that shoulder surface types of approximately the same width did not influence speeds significantly.

Vehicle Placements

The average lateral positions of free-moving passenger cars and of passenger cars meeting other vehicles are shown in figure 1. Free-moving passenger cars maintained an average lateral position progressively farther from the centerline of the highway as the shoulder was widened and improved in type. On sections with 6- to 10-foot gravel shoulders, the average

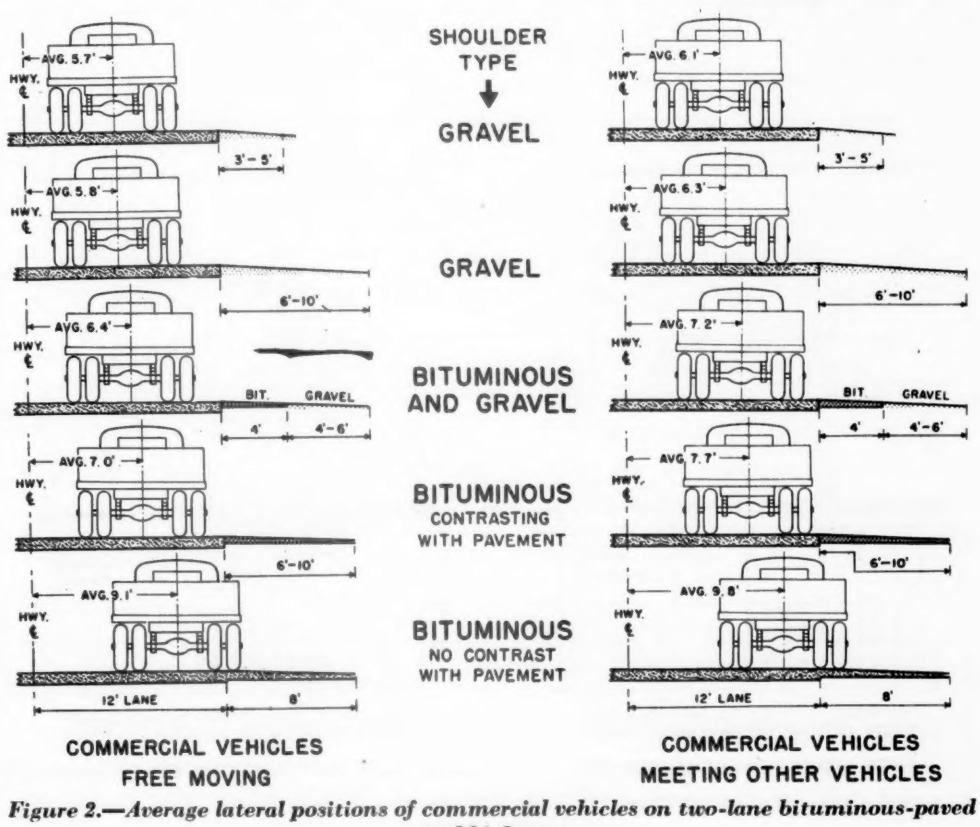

Figure 2.—Average lateral positions of commercial vehicles on two-lane bituminous-paved rural highways.

position was 0.4 foot farther to the right than on sections with 3- to 5-foot gravel shoulders. On sections with 4-foot contrasting bituminous shoulders, and with an additional 4 to 6 feet of gravel outside the paved portion, the average position was 0.4 foot farther to the right than

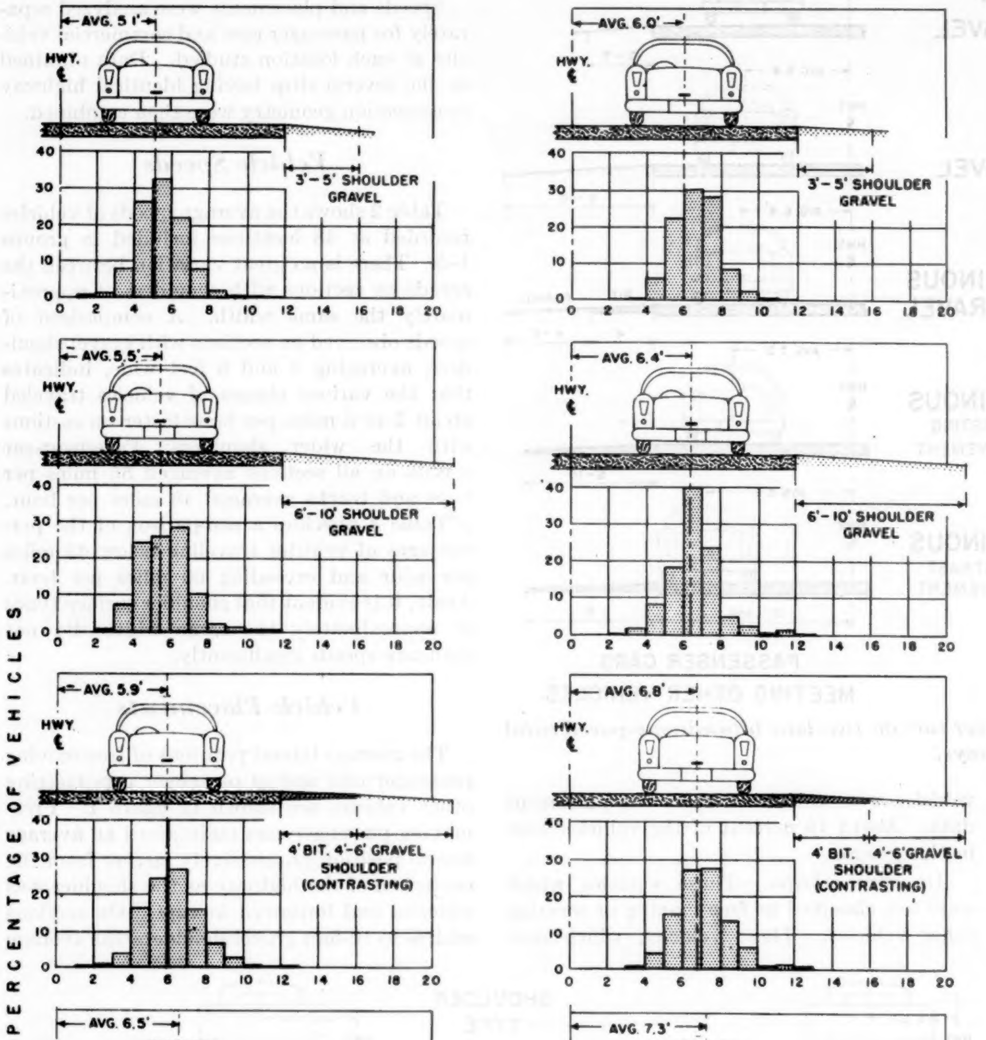

Figure 3.—Distribution of lateral positions of passenger cars on two-lane bituminous-paved rural highways.

on the sections with 6- to 10-foot gravel shoulders. In this progressive trend, the greatest difference in the average lateral position occurred between sections with matching shoulders and traffic lanes as compared with sections with contrasting shoulders and traffic lanes. The difference between the lateral positions of free-moving passenger cars for these two groups was 1.5 feet.

Some highway engineers have expressed the view that an ideal cross section might be one where the average free-moving vehicle travels in the same path as vehicles meeting oncoming traffic. In other words, traffic would assume a certain lateral position and maintain that position even when meeting oncoming vehicles. It is noted in figure 1, however, that free-moving passenger cars travel 0.8 or 0.9 foot nearer the centerline than passenger cars meeting other vehicles for all shoulder widths and types studied.

As indicated in figure 2, free-moving commercial vehicles on the average traveled 0.4 to 0.5 foot closer to the centerline than those meeting other vehicles on sections with gravel shoulders; on the other sections, the difference was 0.7 to 0.8 foot.

Figure 3 shows the distribution of lateral positions of free-moving passenger cars and passenger cars meeting other vehicles. This illustration is similar to figure 1, except that it shows the distribution of the lateral positions. The positions were determined by measuring the distance from the center of the car to the centerline of the pavement. A value less than 3 feet from the centerline of the highway indicates that the car was encroaching on the left traffic lane. Similarly, a value greater than 9 feet indicates that the car was encroaching on the shoulder area. Figure 3 shows that on two-lane, 24-foot bituminous pavements with gravel shoulders the lateral positions are concentrated mainly within a 3-foot strip in the center of the traffic lane. On sections with 8-foot matching shoulders, the lateral positions are distributed over a distance of about 6 feet.

Encroachment on left lane and on shoulder

The percentages of vehicles encroaching on the left lane and on the shoulder are shown in table 4. It is evident that the type of shoulder does not materially affect the percentages of vehicles straddling the centerline. In general, few vehicles straddle the centerline, and this is particularly true of vehicles meeting other traffic. This indicates that 12-foot lanes are adequate for two-lane rural highways and supports the findings of a previous study.[2]

Shoulder use, as shown in table 4, is definitely related to the shoulder type. As might be expected, vehicles meeting other traffic encroach on the shoulder to a larger extent than vehicles in any of the other groups. Commercial vehicles make greater use of the shoulders than passenger cars. The degree of encroachment on the shoulder under various conditions is illustrated in figure 4.

Encroachment on the shoulder area was greatest on sections with matching pavement

[2] *Effect of roadway width on vehicle operation*, by A. Taragin. PUBLIC ROADS, vol. 24, No. 6, Oct.-Nov.-Dec. 1945.

Table 4.—Percentages of vehicles straddling centerline or traveling on shoulders of two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and shoulders of various surface types and widths

Vehicle classification	Percentage of vehicles traveling at indicated lateral positions on two-lane highways with—				
	Gravel shoulders, 3-5 feet	Gravel shoulders, 6-10 feet	Bituminous and gravel shoulders, 8-10 feet [1]	Bituminous shoulders, 6-10 feet [2]	Bituminous shoulders, 8 feet [3]
Vehicles Straddling Centerline					
Passenger cars:	Percent	Percent	Percent	Percent	Percent
Free moving	1.3	0.6	1.0	0.4	0.4
Meeting other vehicles	.2	.1	.3	.2	0
All other passenger cars	2.8	1.4	1.2	.9	.9
Commercial vehicles:					
Free moving	2.5	3.9	4.9	.9	1.3
Meeting other vehicles	0	3.1	1.0	1.5	.5
All other commercial vehicles	3.1	2.0	5.1	1.8	1.4
Vehicles Traveling on Shoulder					
Passenger cars:					
Free moving	0.7	1.7	3.8	11.6	33.8
Meeting other vehicles	3.4	3.3	10.0	19.8	55.1
All other passenger cars	.6	1.9	5.8	12.1	38.1
Commercial vehicles:					
Free moving	6.1	3.9	16.4	27.8	67.3
Meeting other vehicles	8.3	10.5	31.6	40.7	78.4
All other commercial vehicles	4.6	5.7	24.5	30.8	68.8

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

and shoulders. On these sections nearly 80 percent of the trucks meeting other vehicles traveled partly on the shoulder. On the basis of these findings, it would appear that there is justification for placing a uniform subgrade and asphalt pavement over the entire width of the roadway. Arizona, for example, has followed this practice for a number of years. The data in figure 4 indicate, however, that if there is a contrast between traffic lanes and shoulders, the structural strength of the entire width of the shoulder need not be as great as that of the traveled lane. Shoulder encroachment on sections with contrasting shoulders is one-third to one-half that of sections with matching pavement and shoulders.

Clearances Between Meeting Vehicles

Figure 5 shows the clearances between bodies of passenger cars meeting other passenger cars for the various types and widths of shoulders. The percentage of vehicles having clearances of less than 4 feet was very small. On sections with narrow gravel shoulders, 3 to 5 feet, the average clearance between bodies of passenger cars meeting other passenger cars was 5.8 feet. The average clearances increased with the wider and higher type shoulders. On sections with matching pavement and shoulders, the clearances between bodies of meeting passenger cars averaged 11.1 feet, with 94 percent of the vehicles having clearances of 6.0 feet or more. It is noteworthy that clearances of 10 feet or more were recorded in 63 out of 100 observations on matching pavement and shoulder sections, whereas on sections with gravel shoulders, clearances of 10 feet or more were recorded in 3 out of 100 observations.

A cumulative distribution of clearances between bodies of passenger cars meeting commercial vehicles is shown in figure 6. Clearances were least on sections with gravel shoulders. On sections with 3- to 5-foot gravel shoulders, 16 percent of the vehicles had clearances of 4 feet or less. On sections with 4-foot bituminous-paved shoulders and 4- to 6-foot gravel surfaces outside the paved portion, only 3 percent of the clearances were below 4 feet. On sections where the shoulders were paved with 6 to 10 feet of contrasting bituminous material, the clearances were only slightly greater than on sections with 4 feet of bituminous material in combination with 4 to 6 feet of gravel. It is obvious that on sections with matching bituminous shoulders, the clearances between passenger cars meeting commercial vehicles are greater than needed for safe operation.

Average clearances between the bodies of commercial vehicles meeting other commercial vehicles were obtained at several locations and the results are shown in the last column of table 5. Although the sample was small, it is interesting to note that only on sections with contrasting pavement and shoulders were the clearances between the bodies of meeting commercial vehicles about the same as for passenger cars.

Sections with oil-penetration shoulders covered with crushed gravel were included with gravel shoulder sections of the same width. Driver behavior followed the same pattern for both treated and untreated gravel shoulders. This was to be expected because drivers could not distinguish between the two types.

Relation Between Speed and Placement

On sections of highways having the usual 12-foot traffic lanes with grass or gravel shoulders, studies heretofore have shown that there is very little relation between the speeds of vehicles and their lateral positions in the traffic lane. Because of the many sections with paved shoulders included in this study, an effort was made to determine whether any relation exists between the speed of a vehicle and its lateral position on the highway.

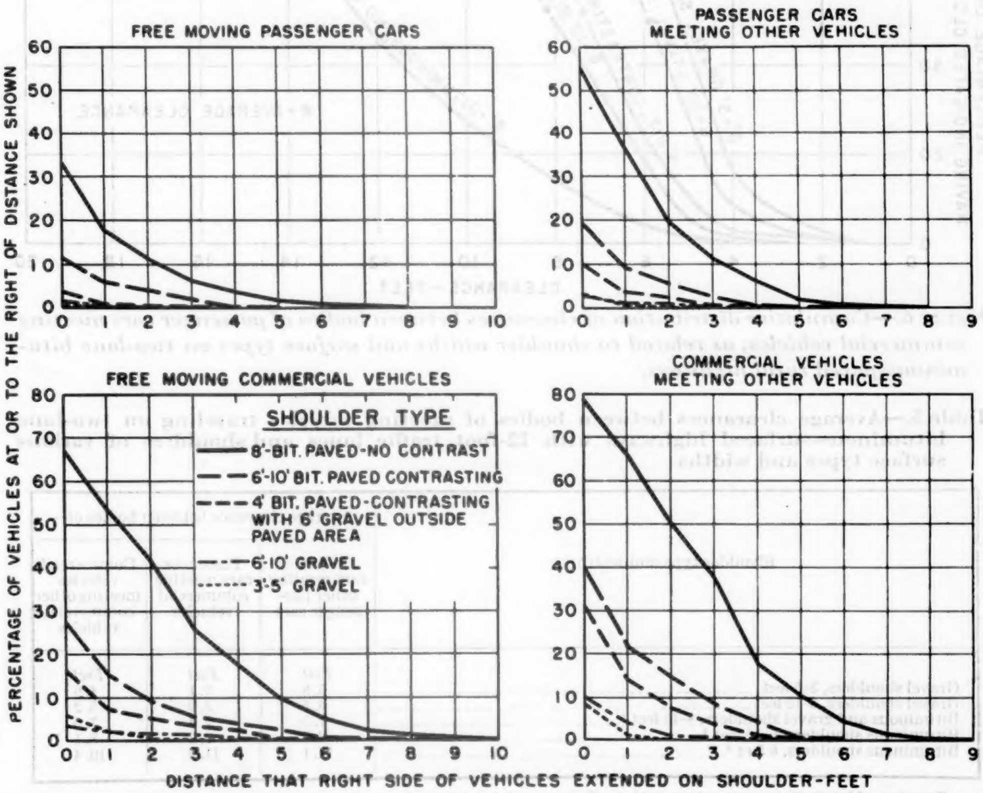

Figure 4.—Shoulder encroachment of passenger cars and commercial vehicles on two-lane bituminous-paved rural highways.

A typical illustration of the results is shown in figure 7. This figure shows the average position of free-moving passenger cars traveling at various speeds on two-lane rural roads with various shoulder widths and types. On sections with gravel shoulders, there was only a very slight tendency for the slower moving vehicles to travel closer to the shoulder area than the faster moving vehicles. The tendency was much greater, however, on paved shoulder sections. On sections where the pavement and shoulders were uniform in appearance, the lateral position of the slowest group of drivers was more than 2.0 feet closer to the shoulder than the position of the fastest group. In other words, the slower moving passenger cars utilized the full width of paving to a greater degree than the faster vehicles. Relations similar to those shown in figure 7 for free-moving passenger cars were found to exist for other groups of passenger cars and commercial vehicles.

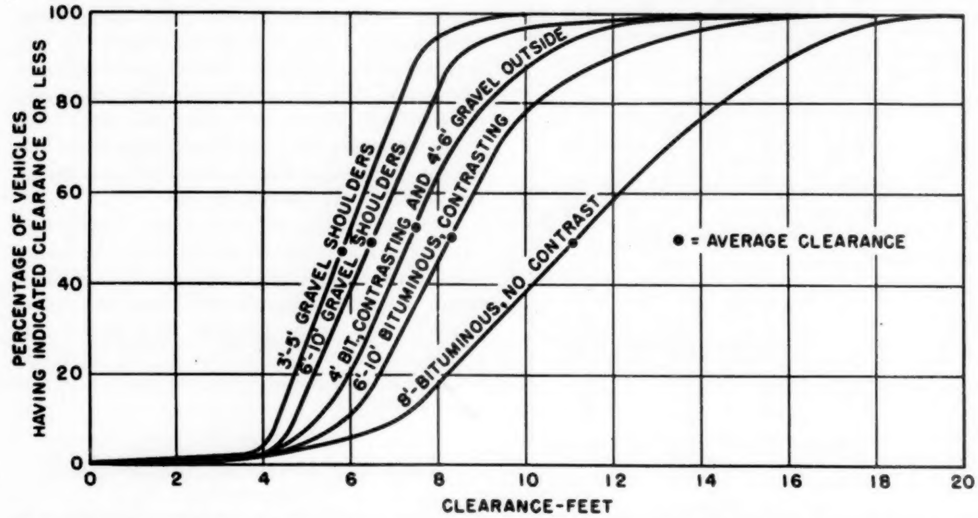

Figure 5.—Cumulative distribution of clearances between bodies of meeting passenger cars, as related to shoulder widths and surface types on two-lane bituminous-paved rural highways.

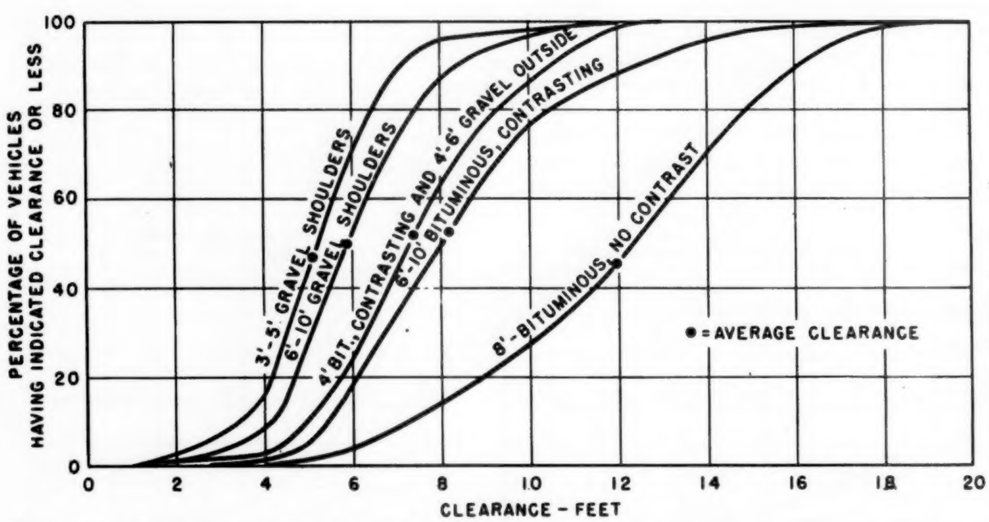

Figure 6.—Cumulative distribution of clearances between bodies of passenger cars meeting commercial vehicles, as related to shoulder widths and surface types on two-lane bituminous-paved rural highways.

Table 5.—Average clearances between bodies of meeting vehicles traveling on two-lane bituminous-surfaced highways with 12-foot traffic lanes and shoulders of various surface types and widths

Shoulder type and width	Average clearance between bodies of—		
	Passenger cars meeting other passenger cars	Passenger cars meeting commercial vehicles	Commercial vehicles meeting other commercial vehicles
	Feet	*Feet*	*Feet*
Gravel shoulders, 3–5 feet	5.8	5.1	4.6
Gravel shoulders, 6–10 feet	6.5	5.9	5.2
Bitiminous and gravel shoulders, 8–10 feet [1]	7.5	7.4	7.6
Bituminous shoulders, 6–10 feet [2]	8.3	8.2	8.1
Bituminous shoulders, 8 feet [3]	11.1	11.9	10.4

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

Passing Practices

In addition to speed and placement studies, passing maneuvers were observed over a one-half mile length of highway at 27 locations. The observers classified each passing maneuver according to the relative transverse and longitudinal positions of the passed and passing vehicles, as shown in table 6. The number of passings per mile per hour on sections with combination-type shoulders (bituminous material and gravel) was the same as on sections with contrasting bituminous shoulders and traffic lanes. On bituminous-paved sections with matching shoulders and traffic lanes, the number of passings performed, reduced to a common traffic volume, was about 30 percent higher than on the other two cross sections.

It appears therefore that matching traffic lanes and shoulders tend to encourage passing maneuvers. Three-lane operation was almost nonexistent on sections having shoulders surfaced with a combination of gravel and bituminous material.

Concrete Pavement Operation

As previously mentioned, shoulder usage was less on bituminous pavements with contrasting shoulders and traffic lanes than on sections with matching shoulders and traffic lanes. The shoulder usage was further reduced on concrete pavements with bituminous shoulders, and still further reduced on sections with grass or gravel shoulders.

Speed and placement data were recorded on six level tangent sections of two-lane, 24-foot concrete pavements. On three of these sections the pavement was flanked by 9-foot bituminous-paved shoulders, and on the other three sections the shoulders were grass or gravel measuring 8 to 10 feet wide. Table 7 compares the results obtained on sections having bituminous shoulders with those obtained on sections with grass or gravel shoulders. The latter group of sections is identified in table 7 as gravel shoulders, since results of an earlier study [3] as well as the results of the present study indicate that well-maintained grass shoulders have the same effect on the lateral position of moving vehicles as well-maintained gravel shoulders.

The results shown in table 7 indicate that the speeds were about the same on the road sections with gravel shoulders as on the sections with bituminous shoulders. Passenger cars traveled about one foot closer to the bituminous shoulders than to the gravel shoulders, while commercial vehicles maintained about the same average lateral position for both shoulder types.

[3] See footnote 2, p. 200.

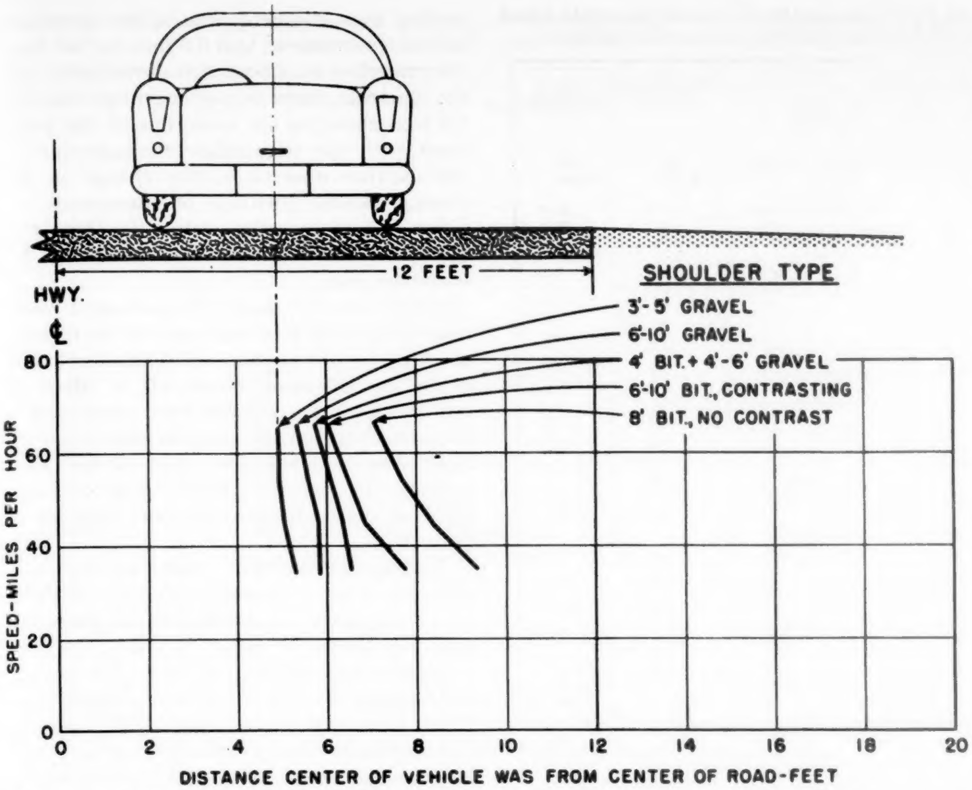

Figure 7.—Average lateral positions of free-moving passenger cars traveling at various speeds on two-lane bituminous-paved rural highways.

Although the percentages of vehicles straddling the centerline were small for either type of shoulder, passenger cars straddled the centerline on sections with gravel shoulders more often than on sections with bituminous shoulders. On the other hand, results show an opposite trend for commercial vehicles, although the difference was slight.

A comparison of the results shown in table 7 for concrete pavements and similar results for bituminous pavements, shown in figures 1–2, reveals that bituminous shoulders adjacent to the concrete traffic lanes were more effective in confining vehicles to the traffic lane than bituminous pavements with contrasting bituminous shoulders. Vehicle operation, as measured by lateral positions and clearances between vehicles, on two-lane, 24-foot concrete pavements with 9-foot bituminous shoulders was about the same as that for two-lane, 24-foot bituminous pavements with a combination of 8- to 10-foot bituminous and gravel shoulders. It is evident that the greater the contrast in appearance between the traffic lane and the paved shoulder, the better the positioning of the moving vehicle within the traffic lane.

Substandard Pavement Widths

Although 11-foot traffic lanes are not now the standard width for primary two-lane highways, there is a considerable mileage of such roads. In conjunction with the studies on sections with 12-foot lanes, four locations with 11-foot lanes were studied in three States. These pavements were flanked by 6- to 8-foot bituminous shoulders. One of the locations in Oregon had red paved shoulders adjacent to the traffic lanes. Another location, also in Oregon, had black shoulders adjacent to a red pavement. Red aggregate with an asphaltic binder was used to get this color contrast. On the other two locations in California and Washington, the shoulders and traffic lanes were bituminous surfaced with a definite contrast in appearance.

Vehicle speeds on these 22-foot pavements were about the same as for the other sections of two-lane roads with wider surfaces. Lateral positions with respect to the highway centerline and clearances between meeting vehicles on these 22-foot sections also followed a similar pattern in relation to two-lane, 24-foot bituminous pavements with 6- to 10-foot gravel shoulders. It appears therefore that paved shoulders adjacent to a two-lane, 22-foot pavement increase the effective surface width about 2 feet.

Vehicle speeds and lateral positions on the sections in Oregon with red and black combinations of pavements and shoulders were similar regardless of the order of color. Furthermore, there was no significant difference between traffic operations on sections with red and black combinations and those with contrasting shoulders and lanes of bituminous material.

Effectiveness of Edge Striping

Special studies of the effect of edge stripes on vehicle speeds and placements were conducted in conjunction with the studies of matching pavement and shoulder sections. These pavements appeared to be two 20-foot lanes without shoulders. Edge stripes were studied in cooperation with the State highway departments of Arizona, Utah, and Idaho. Standard centerline markings were used in all three States. These consisted of 4-inch-wide white reflectorized dashes, 15 feet long with 25-foot spaces, as illustrated in figure 8.

Two sections in Arizona, both of which were level tangents and on the same highway, were studied simultaneously. One section had no edge stripes; the other had 4-inch white reflectorized dashes, 7 feet long with 33-foot spaces. The dashes were 1 foot from the pavement edge or 19 feet from the centerline of the roadway.

In Utah, the studies were conducted at the same location before and after the edge stripes were painted. The edge stripes were 4-inch solid yellow reflectorized, and were placed 1.5 feet from the pavement edge or 18.5 feet from the centerline. Two locations were studied, one on a level tangent and the other near the top of a 3-percent grade about 2,000 feet long.

The speed data shown in table 8 indicate

Table 6.—Passing maneuvers observed on one-half mile sections of two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and bituminous-paved shoulders

Traffic data	Shoulder surface types and widths		
	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders, 8 feet [3]
Number of locations studied	3	11	13
Total hours of study	24	87	95
Average traffic volume ... vehicles per hour	210	210	120
Number of passings per half mile:			
Two vehicles abreast:			
Passing vehicle, right of centerline	------	------	4
Passing vehicle, straddling centerline	329	1,072	385
Passing vehicle, left of centerline	55	271	252
Three vehicles abreast [4]	2	57	8
Four vehicles abreast	------	2	2
Total passing maneuvers	386	1,402	651
Number of passings per mile per hour	32.2	32.2	13.7
Equivalent number of passings per mile per hour for—			
Traffic volumes of 210 vehicles per hour	32.2	32.2	[5] 42.0
Traffic volumes of 120 vehicles per hour	[5] 10.5	[5] 10.5	13.7

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.
[4] Two vehicles traveling in one direction.
[5] Computation based on the assumption that the number of passings varies as the square of the hourly traffic volume.

Table 7.—Summary of speeds and placements on two-lane portland cement concrete rural highways with 12-foot traffic lanes and 8- to 10-foot bituminous or gravel shoulders

Items of information	Bituminous shoulders	Gravel shoulders
Number of locations studied	3	3
Average traffic volume ... vehicles per hour	200	310
Number of vehicles studied	4,928	5,000
Average speed:		
Passenger cars ... miles per hour	51.3	50.9
Commercial vehicles ... do	45.1	42.6
Average lateral position: [1]		
Passenger cars:		
Free moving ... feet	6.0	5.2
Meeting other vehicles ... do	6.7	5.9
All other passenger cars ... do	6.2	5.4
Commercial vehicles:		
Free moving ... do	6.2	5.9
Meeting other vehicles ... do	6.5	6.6
All other commercial vehicles ... do	6.2	6.0
Vehicles straddling centerline:		
Passenger cars:		
Free moving ... percent	.4	1.6
Meeting other vehicles ... do	.1	.5
All other passenger cars ... do	.6	1.1
Commercial vehicles:		
Free moving ... do	.6	-------
Meeting other vehicles ... do	-------	-------
All other commercial vehicles ... do	4.1	1.6
Vehicles encroaching on shoulder:		
Passenger cars:		
Free moving ... do	3.6	-------
Meeting other vehicles ... do	5.2	-------
All other passenger cars ... do	3.5	(2)
Commercial vehicles:		
Free moving ... do	8.5	1.0
Meeting other vehicles ... do	10.3	5.0
All other commercial vehicles ... do	7.3	1.3
Clearance between bodies:		
Passenger cars meeting other vehicles ... feet	7.1	5.5
Commercial vehicles meeting other vehicles ... do	6.3	5.7

[1] Distance center of vehicle was from centerline of pavement.
[2] Less than 0.05 percent.

that edge stripes caused an increase in speeds in some cases and a decrease in speeds in others. The wide disparity in the results makes it impossible to draw any conclusion as to the effect of edge stripes on vehicle speeds. A study of the lateral positions of vehicles offers a much more conclusive measure of the effect of edge stripes, the results of which are also included in table 8.

The edge stripe used in Arizona caused drivers of passenger cars to travel during the day 0.1 foot closer to the centerline of the pavement than when there was no stripe, and at night they traveled 0.4 foot farther from the centerline. The edge stripe used in Utah had practically no effect on passenger car placements at night. During the day, however, the average lateral position was 0.6 foot farther from the centerline of the pavement on the level tangent and 0.9 foot farther from the centerline on a 3-percent downgrade. On the 3-percent upgrade, passenger cars traveled 1.0 foot closer to the centerline of the pavement with the edge stripe than without it. For daytime operations, the change in the average lateral positions of commercial vehicles caused by edge stripes in these two States was about the same as the change for passenger cars.

Along with the study of speeds and placements, an evaluation was made of the relation of striping to shoulder encroachment. Because the pavement consisted, in effect, of two 20-foot lanes, vehicles were considered to be encroaching on the shoulder area when the right side extended more than 12 feet from the centerline of the roadway. This encroachment is shown in the last four columns of table 8.

During the daytime, passenger cars observed in Arizona encroached on the shoulder about the same extent whether the pavement edge was striped or not. At night, however, a considerably larger percentage of cars used the shoulder as a result of edge striping. In Utah, the stripe on the level tangent section had very little effect on shoulder encroachment of passenger cars traveling during the day or night. On the 3-percent downgrade, the stripe caused passenger cars to encroach on the shoulder more during the day and less at night. On the upgrade, the stripe reduced shoulder encroachment during the day and night.

As was mentioned earlier, nearly 80 percent of the trucks encroached on the shoulder area of matching shoulder and traffic lane sections.

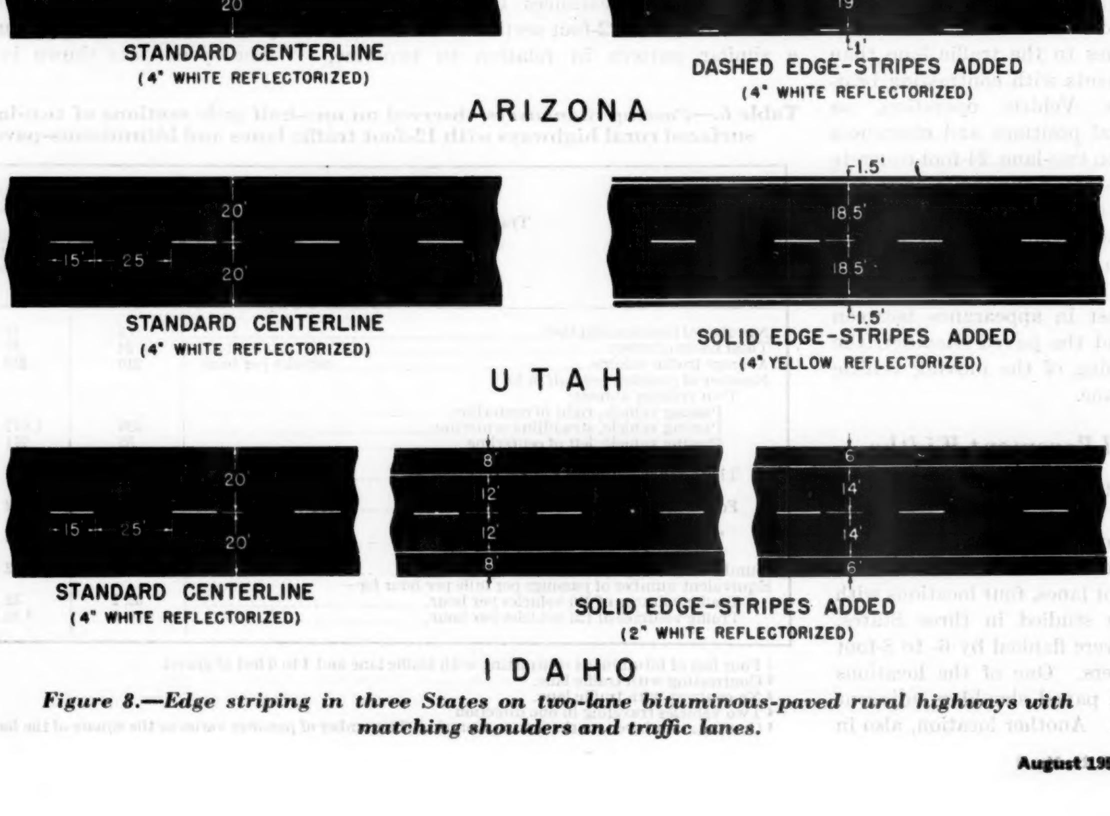

Figure 8.—Edge striping in three States on two-lane bituminous-paved rural highways with matching shoulders and traffic lanes.

Table 8.—Effect of edge striping on speeds and lateral positions of vehicles and their encroachment on shoulders of two-lane bituminous pavements with full-width paved shoulders having the same appearance as the traffic lanes

State	Gradient	Edge stripe				Change in speed resulting from edge striping [2]		Change in lateral position resulting from edge striping [3]		Encroachment on shoulder area [4]			
										Daytime		Nighttime	
		Color	Type	Width	Distance from edge [1]	Daytime	Nighttime	Daytime	Nighttime	Without stripe	With stripe	Without stripe	With stripe
PASSENGER CARS													
	Percent			*Inches*	*Feet*	*M. p. h.*	*M. p. h.*	*Feet*	*Feet*	*Percent*	*Percent*	*Percent*	*Percent*
Arizona	0	White	Dashed	4	1.0	+7.3	+9.5	−0.1	0.4	43.4	45.2	3.7	21.7
Utah	0	Yellow	Solid	4	1.5	+2.2	−2.5	.6	−.1	32.1	34.8	22.3	21.3
Do	−3.0	do	do	4	1.5	−3.9	+.9	.9	−.1	44.5	57.0	8.1	3.8
Do	+3.0	do	do	4	1.5	+4.7	+3.8	−1.0	−.1	45.7	30.8	11.4	8.9
Idaho	0	White	do	2	8.0	−9.2	−9.1	−1.9	−2.5	60.7	24.9	(5)	23.7
Do	0	do	do	2	6.0	−6.9	−4.7	−1.6	−2.3	60.7	31.0	(5)	25.9
COMMERCIAL VEHICLES													
Arizona	0	White	Dashed	4	1.0	+6.2	(5)	−0.2	0.1	97.6	98.5	100.0	66.7
Utah	0	Yellow	Solid	4	1.5	+2.3	−3.3	.5	.2	56.1	65.0	48.6	53.3
Do	−3.0	do	do	4	1.5	−2.3	−4.5	.9	1.3	46.0	74.2	6.2	35.0
Do	+3.0	do	do	4	1.5	+5.6	+7.0	−1.1	−.1	86.4	69.1	45.4	43.7
Idaho	0	White	do	2	8.0	−4.6	−4.2	−1.7	−2.0	86.8	42.9	(5)	46.4
Do	0	do	do	2	6.0	−1.4	0	−1.7	−2.6	86.8	61.0	(5)	51.3

[1] Distance toward centerline from edge of the full-width paving (12-foot traffic lane plus 8-foot shoulder, or a total of 20 feet).
[2] Average speed observed on striped section compared with that on unstriped section.
[3] Average lateral position on striped section compared with that on unstriped section. Minus values indicate that the average position on striped section was closer to the centerline of the highway than on the unstriped section.
[4] Percentages of vehicles with right side extending more than 12 feet from centerline of roadway.
[5] No data available.

In Arizona, edge stripes were not effective in reducing encroachment except at night. The tests made in Utah indicated that trucks made greater use of the shoulders after striping, except on the one section with the 3-percent upgrade.

The studies of edge striping in Idaho were conducted on one level tangent section of highway. The original 4-mile section was divided into two sections of equal length. Two-inch-wide solid reflectorized stripes were painted on one section 8 feet from the pavement edge or 12 feet from the centerline, as shown in figure 8. On the other section the stripe was painted 6 feet from the edge or 14 feet from the centerline.

No firm conclusion can be drawn as to the effect of these edge stripes on vehicle speeds, but vehicles traveled considerably closer to the centerline of the pavement as a result of striping. The effect was greater at night than during the day, and generally speaking, stripes placed 8 feet from the edge of the shoulder influenced drivers more than those placed at 6 feet. As a consequence of the tendency to crowd the centerline, there was less travel on the shoulders.

Although fewer vehicles in Idaho traveled during the day on the shoulder area of the section with stripes placed 8 feet from the shoulder edge as compared with those placed at 6 feet, some vehicles tended to use the 8 feet to the right of the stripe as a traffic lane. In effect, the pavement was used as a four-lane undivided highway except that the distribution of traffic between lanes was reversed. In other words, the percentage of vehicles traveling in the 12-foot lane approximated the percentage that normally would travel in the right lane of a four-lane undivided highway with an equivalent volume of traffic.

Table 9.—Vehicle speeds related to special signs and edge stripes on a two-lane bituminous pavement in Oregon

Conditions of study [1]	Average speed	Vehicles traveling over 60 m. p. h.	Vehicles traveling under 40 m. p. h.
PASSENGER CARS			
Daytime:	*M. p. h.*	*Percent*	*Percent*
Normal conditions	61.3	60.0	1.8
Signs only	58.3	43.0	3.3
Signs and edge stripes	61.3	60.5	1.4
Edge stripes only	57.9	46.5	3.0
Nighttime (edge stripes only)	56.7	41.5	1.8
COMMERCIAL VEHICLES			
Daytime:			
Normal conditions	53.2	18.0	3.2
Signs only	51.3	13.0	5.6
Signs and edge stripes	52.7	13.2	1.3
Edge stripes only	49.6	5.4	3.9
Nighttime (edge stripes only)	49.0	6.3	1.4

[1] Legend on signs: NO TRAVELING ON PAVED SHOULDERS. Edge stripes were 4-inch solid yellow reflectorized, 13 feet from centerline of roadway.

Special Studies in Oregon

A level tangent section of highway in Oregon was selected for a study of special edge stripes and signs. The section of highway consisted of two 12-foot bituminous lanes flanked by 10-foot shoulders—4 feet of contrasting bituminous material and 6 feet of gravel.

Speed and placement data were recorded at the same point on the highway under the conditions shown in table 9. The average speeds of passenger cars and commercial vehicles for the five conditions of study and the percentages of vehicles traveling above 60 miles per hour and below 40 miles per hour are included in this table. Under normal conditions, the average speeds were 61.3 miles per hour for passenger cars and 53.2 miles per hour for trucks. The lowest average speed during the daytime was observed when only the edge stripes were used, whereas the lowest percentage of passenger cars traveling over 60 miles per hour during the day was observed when only the signs were used. The per-

(*Continued on page 215*)

Table 10.—Lateral positions of vehicles related to special signs and edge stripes on a two-lane bituminous pavement in Oregon

Conditions of study [1]	Lateral position [2]	Vehicles encroaching on shoulders	Clearance between bodies of meeting vehicles
PASSENGER CARS			
Daytime:	*Feet*	*Percent*	*Feet*
Normal conditions	6.6	10.7	8.1
Signs only	6.4	8.4	7.9
Signs and edge stripes	6.1	4.3	7.2
Edge stripes only	6.1	5.4	7.0
Nighttime (edge stripes only)	6.0	3.7	7.6
COMMERCIAL VEHICLES			
Daytime:			
Normal conditions	7.2	41.3	8.4
Signs only	6.6	30.7	7.7
Signs and edge stripes	6.3	18.2	6.2
Edge stripes only	6.2	13.1	7.2
Nighttime (edge stripes only)	6.1	6.3	6.3

[1] Legend on signs: NO TRAVELING ON PAVED SHOULDERS. Edge stripes were 4-inch solid yellow reflectorized, 13 feet from centerline of roadway.
[2] Distance from center of vehicle to centerline of roadway.

The AE-55 Indicator Used for Determining Air Content of Concrete

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB, Highway Physical Research Engineer

THE AE–55 air indicator or Chace air meter is a pocket-sized device intended for use in the field to estimate the air content of plastic concrete. This apparatus was developed by L. M. Chace, a consulting engineer of North Bridgeton, Maine. A number of State highway departments and other organizations have purchased this device and are using it experimentally in the field. They are interested in it because of its low cost, rapidity of operation, and convenience to the engineer in the field.

Tests were made in the laboratory of the Bureau of Public Roads to obtain information on the accuracy and dependability of this apparatus. The air content of a large number of concrete mixes prepared in the laboratory was determined using this indicator, and the results were compared with those obtained by the gravimetric and the pressure methods, AASHO methods T 121 and T 152, respectively.

The AE–55 air indicator shown in figure 1 consists of two parts. One part is a small cylinder of Pyrex glass about 1 inch in diameter and 3 inches long which tapers at one end to a stem or tube about ¼ inch in diameter and 3 inches in length. This is similar to the filtration crucible holders shown in catalogs of laboratory apparatus. The other part is a rubber stopper with a brass cup mounted on the smaller end. The stopper fits the larger end of the glass cylinder. The brass cup is ¾ inch in diameter and ½ inch in depth, and has a volume of approximately 3.7 ml. When the stopper and cup are inserted into the cylinder, the volume of the latter is about 27 ml. Eleven equally spaced graduations are etched on the stem of the cylinder, each pair indicating a volume of about 0.08 ml.

[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

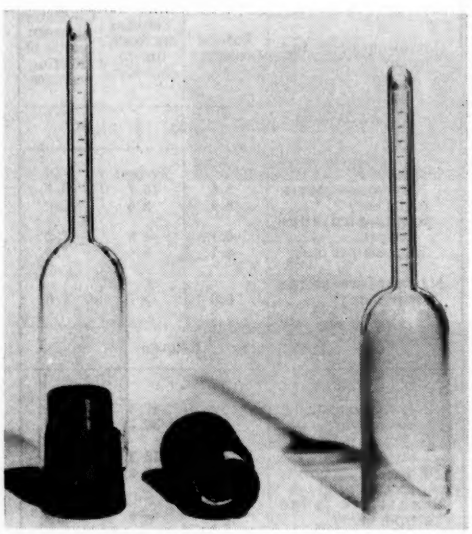

Figure 1.—The AE–55 indicator (assembled and unassembled) used to estimate air content in concrete.

When the concrete tested contains 15 cubic feet of mortar per cubic yard, each graduation on the stem of the indicator represents 1 percent of air in the concrete. If the concrete contains a different amount of mortar, the correction factors given in table 1 are applied. These factors were furnished by the manufacturer of the indicator.

Method of Test

In determining the air content of concrete with the indicator, the following procedure was used: The brass cup was filled with mortar from the concrete excluding particles of sand larger than about ⅒ inch. The mortar was compacted by rodding with a thin, stiff wire [2] and then struck off flush with the top of the cup. The sides of the cup and stopper were cleaned of mortar. The stem end of the cylinder was closed by holding the thumb over the end, and the cylinder was filled with denatured alcohol [3] to the mark on the cylinder. The stopper and cup were then inserted into the cylinder. The indicator was righted, the stopper pressed firmly into the cylinder, and the thumb was removed. The level of the alcohol was brought to the upper graduation on the stem of the cylinder by addition of alcohol or by slight movement of the stopper. When alcohol was added, a small syringe or dropper was used. Care was taken to remove all air bubbles from the cylinder and to keep the stopper seated firmly enough to prevent leaking of the alcohol.

The thumb was again placed over the open end of the stem and the indicator turned gently from a vertical to a horizontal position while the body of the cylinder was tapped with the palm of the hand. Care was taken not to disturb the setting of the stopper. This procedure was continued until all of the mortar had been dispersed into the alcohol and no more air bubbles appeared. As a result, the air in the mortar was replaced by alcohol. The indicator was then held in a vertical position and the new level of the alcohol read to the nearest half graduation on the graduated stem. The stopper was then removed and the indicator washed clean with water. Less than 3 minutes was required to make the test.

[2] The wire in a No. 1 Gem paper clip is suitable.

[3] This alcohol was used instead of water to prevent foaming of the liquid when mortar was added.

This method of determining the air is a volumetric method and is similar in principle to that described in ASTM Method C 173, "Air Content of Freshly Mixed Concrete by the Volumetric Method." In the ASTM method, the air in the fresh concrete is measured by displacing it with a liquid and then determining the volume of the liquid used. The ASTM method is not in common use except where the concrete contains slag or other porous aggregates.

Two other methods for the determination of air in plastic concrete are in general use. In ASTM Method C 231 (AASHO Method T 152), the air is determined by measuring the reduction in volume of the concrete when the latter is held in a closed container and subjected to a definite pressure. This method is used extensively and is considered the most reliable. In ASTM Method C 138 (AASHO Method T 121), the air is determined by calculation from the unit weight of the concrete and the batch weights and specific gravities of the materials used. The latter method is used where a pressure air meter is not available. Where specific gravities and

Table 1.—Conversion factors for the AE–55 air indicator used to correct the indicated air content when the concrete contains other than 15 cubic feet of mortar [1]

Mortar per cubic yard of concrete	Conversion factor [2]	Mortar per cubic yard of concrete	Conversion factor [2]
Cu. ft.		*Cu. ft.*	
10.0	0.67	15.5	1.04
10.5	.70	16.0	1.07
11.0	.73	16.5	1.10
11.5	.76	17.0	1.13
12.0	.80	17.5	1.16
12.5	.83	18.0	1.20
13.0	.86	18.5	1.23
13.5	.90	19.0	1.26
14.0	.93	19.5	1.30
14.5	.96	20.0	1.33
15.0	1.00	----------	--------

[1] Factors furnished by manufacturer of apparatus.

[2] Multiply reading on stem of indicator by conversion factor to obtain correct air content.

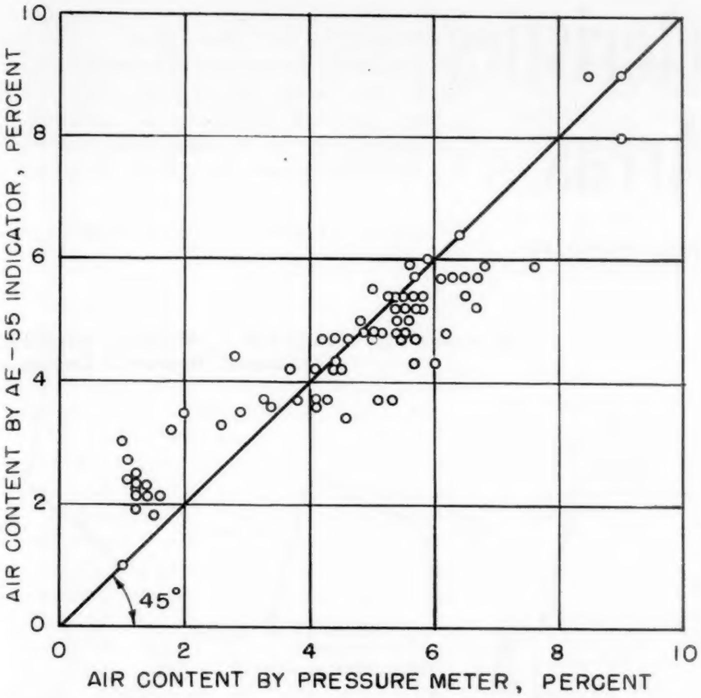

Figure 2.—Air content of concrete as determined by the AE-55 indicator and standard pressure meter.

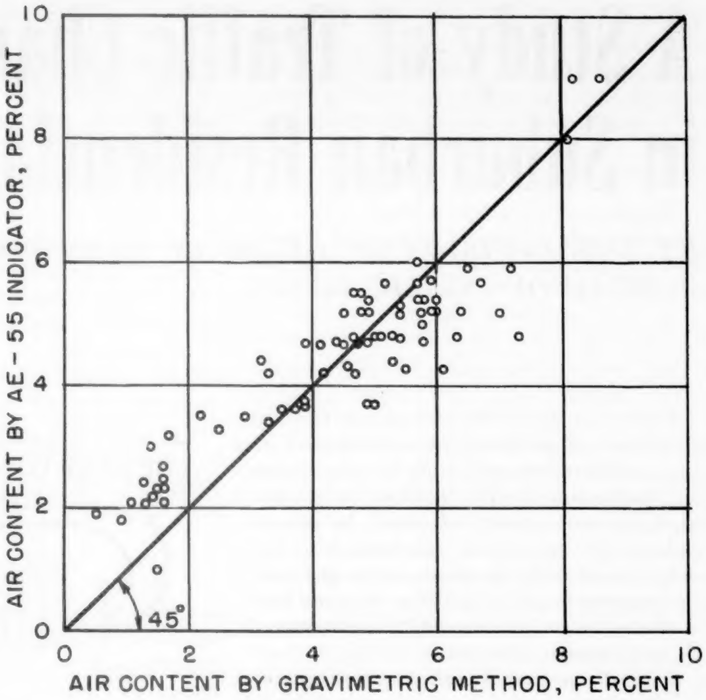

Figure 3.—Air content of concrete as determined by the AE-55 indicator and gravimetric method.

weights are correct and a representative sample is obtained, this method should give an accurate measure of the air content.

Comparison of Three Methods for Measuring Air Content

To determine the suitability of the AE-55 indicator, tests of 84 different concrete mixes were made using this indicator and the pressure and gravimetric methods. The concrete mixes were prepared using different cements and aggregates, and different amounts of air-entraining admixtures to give air contents varying from 1 to 9 percent as determined by the pressure method. Each value reported for the AE-55 indicator is an average of two tests usually made by two operators who generally found results agreeing within 0.5 percent of air. Each value determined by the pressure or gravimetric methods is for a single test. A comparison between the results obtained for each mix by the pressure method and the AE-55 indicator is shown in figure 2. A similar comparison between the results obtained by the AE-55 indicator and the gravimetric method is shown in figure 3.

Table 2.—Average values for different ranges in air content of concrete

Range in air content by pressure meter	Number of samples tested	Average air content measured by—			
		Pressure meter	Gravimetric method	AE-55 indicator	Difference, col. 3 minus col. 5
Pct.		*Pct.*	*Pct.*	*Pct.*	*Pct.*
1.0-1.9	14	1.28	1.36	2.26	−0.98
2.0-2.9	4	2.58	2.70	3.68	−1.10
3.0-3.9	4	3.55	3.65	3.80	−.25
4.0-4.9	14	4.41	4.55	4.24	.17
5.0-5.9	33	5.45	5.36	5.12	.33
6.0-6.9	10	6.42	6.24	5.38	1.04
7.0-7.9	1	7.6	6.5	5.9	1.7
8.0-8.9	4	[1] 8.75	8.28	8.75	.0

[1] Estimated air content—meter can only be read to 8.0 percent.

Average values for several different ranges in air content are given in table 2, together with the difference between the average values for the pressure meter and those for the AE-55 indicator. This table shows good concordance of the average air contents determined by the pressure and gravimetric methods for all values except those above 7 percent. However, the pressure air meter read only to 8 percent, and higher values shown for this meter were estimated. The data show that either method may be used with general assurance that the values obtained indicate accurately the amount of air in the concrete.

The results obtained with the AE-55 indicator did not show as good concordance with those obtained by the pressure meter. For values of air less than 3.0 percent as determined by the pressure meter, the AE-55

(Continued on page 220)

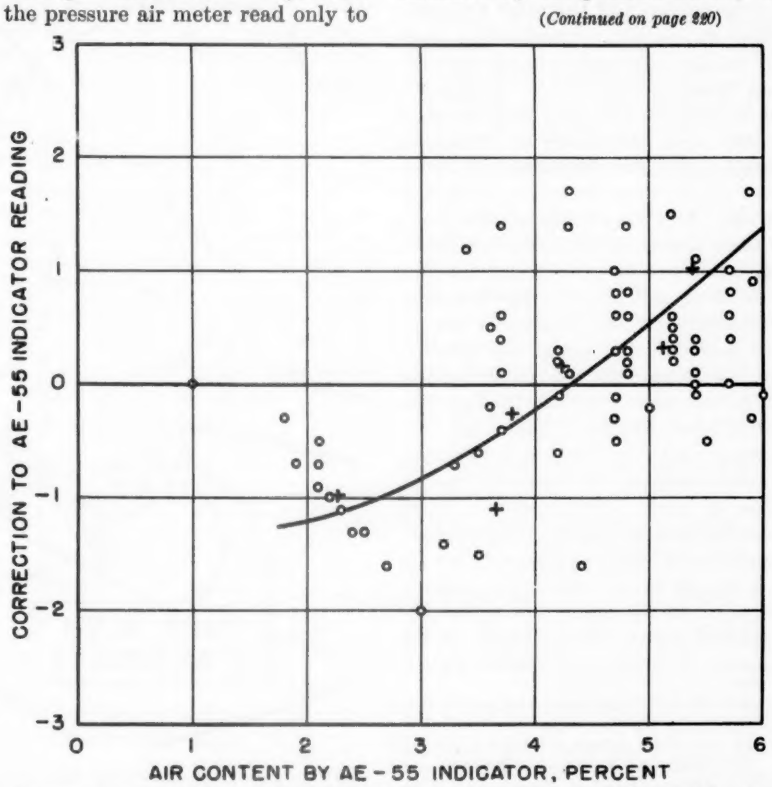

Figure 4.—Correction for AE-55 indicator reading to agree with air content determined by pressure meter.

A Study of Traffic Characteristics in Suburban Residential Areas

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM L. MERTZ, Highway Transport Research Engineer

Suburban land use involving the construction of garden-type apartments and large subdivisions of single homes imposes an enormous traffic burden on existing highway and street systems. In this article, traffic generation resulting from suburban residential development is discussed. Two typical housing facilities, located within the metropolitan area of Washington, D. C., were selected for study.

It was discovered that the two typical housing facilities added as much as 0.8 vehicle per dwelling unit per hour to the adjacent highways during the peak period of traffic flow. Whether the housing facility was a garden-type apartment development or a subdivision of single homes with an equivalent number of dwellings, the traffic generated per dwelling unit followed a similar pattern and volume.

IN RECENT YEARS there has been an exodus of city dwellers to the suburbs, a trend brought about by the widespread ownership of automobiles and the mass construction methods in housing. Emphasis in housing production has changed from the old way of dwellings being constructed one at a time by individual builders to the mass production of hundreds and even thousands of homes by one company. The merchandising of homes has evolved in the same direction as that of the family car. One may select today a mass-produced home that is similar to his neighbor's in the basic construction features and differing only in color, trim, and interior appointments.

The automobile has also drastically altered the environment of the apartment dweller. Apartment developments too have become larger and farther removed from the facilities of the central business district and work centers. No longer is the immediate availability of mass transit facilities the major factor in the location of such developments. These types of residential developments have in turn created problems in highway planning by introducing a change in land use and a concomitant change in travel requirements for the existing highway and street systems. In the past, estimates of future automobile owner-

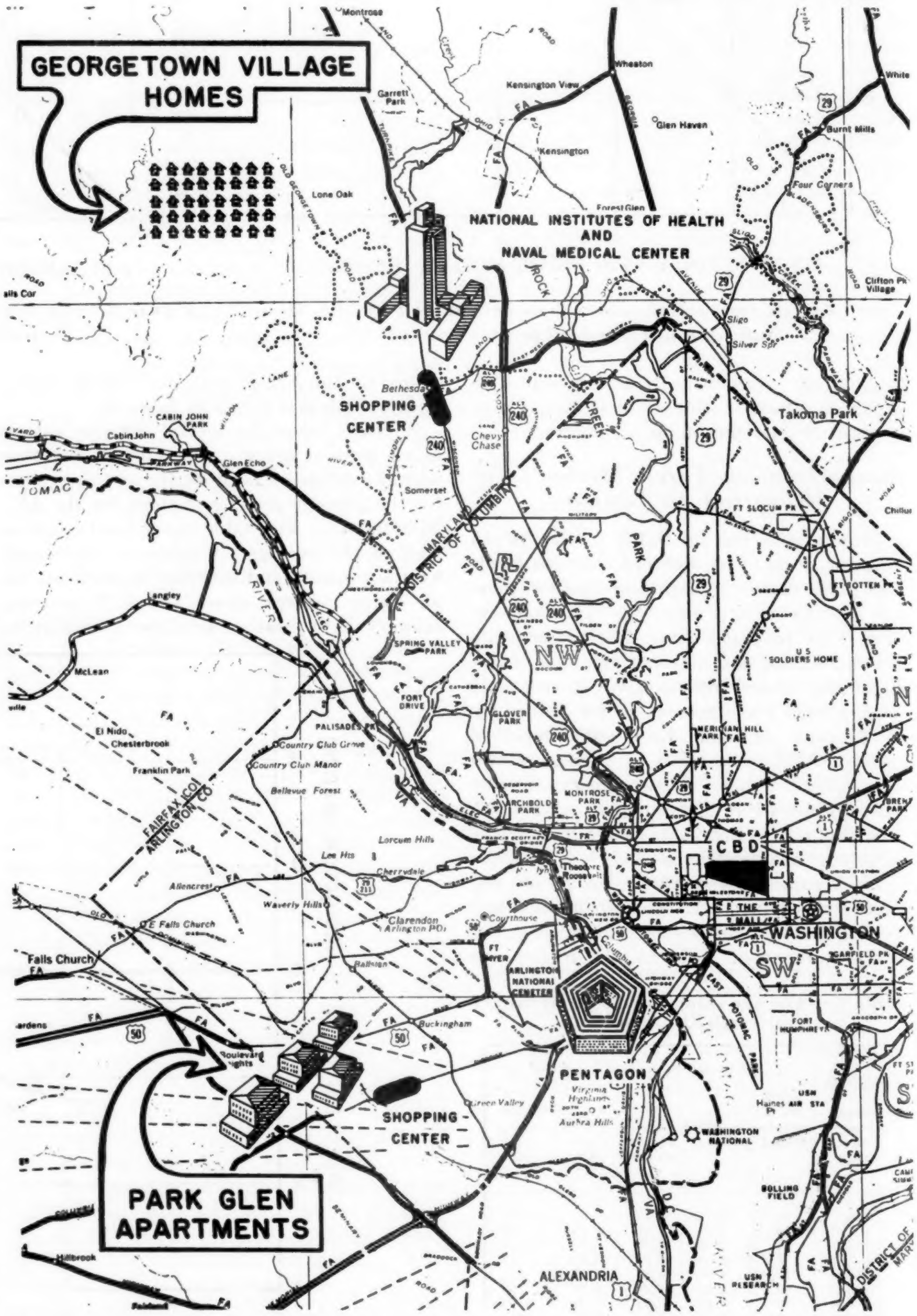

Figure 1.—Map showing the location of the two residential areas studied and their positions with respect to the central business district and other major traffic generating facilities.

[1] The field work and part of the summarizing of the data were done by eight junior highway engineers of the Bureau of Public Roads as a phase of their in-service training in May 1955. Acknowledgment is given the following employees: Frank S. Allison, Daniel H. Brown, Charles M. Moffet, William S. Peterson, Robert A. Quist, Glade W. Roberts, Charles B. Totten, and Sheldon C. Turnidge.

ship and use have generally been too conservative. Consequently, new highway facilities have usually reached their capacity within a few years after construction. Automobile ownership has increased even faster than the increase in population, and the incidence of two-car families is increasing each year.

Factors to Consider in Urban Planning

The Bureau of Public Roads is engaged in continuing research in urban traffic problems. Its research in urban highway planning includes the development and testing of methods of forecasting highway needs. In order to better understand the many complex problems, it has been necessary to assemble much data regarding the travel patterns and habits of the residents of urban areas. Some of the most fruitful sources of information are the urban origin-and-destination traffic studies that have been conducted in most of the major cities. By the use of home interviews, a great wealth of reliable information has been obtained relative to where, why, and how often people make trips. It has been possible to determine the effect of income, population density, and automobile ownership on the number of average daily trips per dwelling unit.

The development of the origin and destination studies involving home interviews has, in recent years, provided engineers and planners with data to help solve today's highway problems, but to estimate future needs they must extrapolate current data. Estimating population growth is a well-developed science, but where this population will locate is not so clearly defined. Forecasting highway needs involves an estimation of future population trends, population density, types of housing, automobile ownership, and vehicle use. The planner must resort to estimates of future growth of urban areas based on some philosophy of expected changes in land use. If reasonably accurate land-use forecasts can be obtained and the traffic that will be generated by these changes can be estimated, the planning engineer is then in a position to apply the data developed by the home-interview type of traffic study to future trends.

Purpose of Study

This article is directed specifically to the feasibility of forecasting traffic demands based upon knowledge of the relation of traffic to residential land use in lieu of the more detailed origin-and-destination traffic studies. In other words, if the forecaster has information about the number of dwelling units per acre, existing and proposed, can he accurately predict the traffic generated by these areas? Also does the type of dwelling unit (apartment or individual home), its distance from the central business district, automobile ownership, and the availability of mass transit facilities appreciably affect traffic generation? Further, can the city planner or engineer predict the effect of changes or the granting of exceptions to the zoning laws on traffic generation?

Sites Selected

The apartment development and the individual home areas studied represent the two extremes of housing facilities in the major cities. Considerable effort was made to select two areas that have quite different characteristics but represent the same economic level. The apartment development has high population density, mass transit facilities available, adjacent shopping facilities, and is located 6 miles from the central business district. The individual home development has relatively low population density, limited transit facilities, no extensive shopping facilities in the immediate neighborhood, and is relatively distant, 12 miles, from the central business district. The location of the two residential areas and their positions with respect to the central business district can readily be ascertained from the map shown in figure 1.

Parkglen is a 216-unit garden-type apartment development located in a built-up area in Arlington County, Virginia. One hundred and ninety-five of the apartments were occupied at the time of the study. The monthly rental rate was $81.50 for a one-bedroom unit and $91.50 for two bedrooms.

The other residential area, Georgetown Village, is a group of 156 single-unit homes in the $13,000–$14,000 price range located at the periphery of the suburban area in Montgomery County, Maryland. One hundred and fifty homes were occupied at the time of the study. Since each of these housing areas was served by one access road, only a small force of observers was needed to record the movement of traffic. The Parkglen apartment development was studied Monday and Tuesday of the first week in May 1955, and Georgetown Village, Monday and Tuesday of the following week. Entering and departing traffic for both areas was observed continuously from 6 a. m. to 10 p. m. An automatic traffic counter was available at Parkglen to verify the recorders' tabulations.

An inventory of passenger cars parked in the areas and their license plates was made during the early morning hours before the studies began. These were assumed to be residents' vehicles. During the study, the recorders noted by hour periods, the State of registration and the license number of each vehicle that entered or left the area for identification as a resident or a nonresident vehicle. During the morning and evening peak-traffic volume periods, however, the information was obtained for quarter-hour intervals. In addition, other pertinent information was recorded such as number of passengers per car, number and type of service trucks, and the number of pedestrians.

Summary of Observations

On the basis of peak traffic volumes recorded during the highest 15-minute interval, it appears that the construction of residential properties similar to Parkglen apartments and Georgetown Village would add about 0.8 vehicle per dwelling unit per hour to the adjacent highway facilities. If the highest hourly volume were used instead of the highest 15-minute volume, the number would be reduced to 0.55 vehicle per dwelling unit per hour. In this study, the average daily number of trips for each of the two residential areas was about five trips per dwelling unit. Other studies have indicated that the traffic patterns of the residential areas considered in this article would not be applicable to densely populated downtown areas where automobile ownership is low and resident parking space is at a premium.

This study has shown that traffic generated per dwelling unit as a result of the construction of either garden-type apartments or single-unit homes follows a similar pattern and volume.

In suburban residential development, highway planning officials must consider the capacity of existing highway and street systems to determine whether there is adequate capacity to accommodate the additional traffic generated through new residential development. If present usage is near capacity, then highway plans must be adjusted accordingly to provide for the increased traffic volume. If additional research confirms the findings of this study, it will be possible to predict future traffic generation of suburban residential land development for a given density of dwelling units.

Automobile Ownership Ratios

Table 1 summarizes the information recorded in the two studies. The automobile ownership ratio of 0.97 per dwelling unit for the apartments and 1.17 for the individual homes follows the same pattern that has been established by current studies based on origin and destination surveys; that is, automobile ownership increases with the distance from the central business district and decreases with a

Table 1.—Vehicular and pedestrian trips recorded for a 2-day period during May 1955

Traffic components	Parkglen apartments (195 occupied dwelling units; 190 resident passenger cars)			Georgetown Village homes (150 occupied dwelling units; 175 resident passenger cars)		
	Number of trips	Percent of total vehicular trips	Average number of trips per dwelling unit per day	Number of trips	Percent of total vehicular trips	Average number of trips per dwelling unit per day
Resident passenger cars	1,136	57.4	2.91	885	59.1	2.95
Nonresident passenger cars	657	33.2	1.68	483	32.3	1.61
Total, passenger cars	1,793	90.6	4.59	1,368	91.4	4.56
Service trucks	186	9.4	.48	128	8.6	.43
Total vehicles	1,979	100.0	5.07	1,496	100.0	4.99
Pedestrian	507		1.30	93		.31
Total	2,486		6.37	1,589		5.30

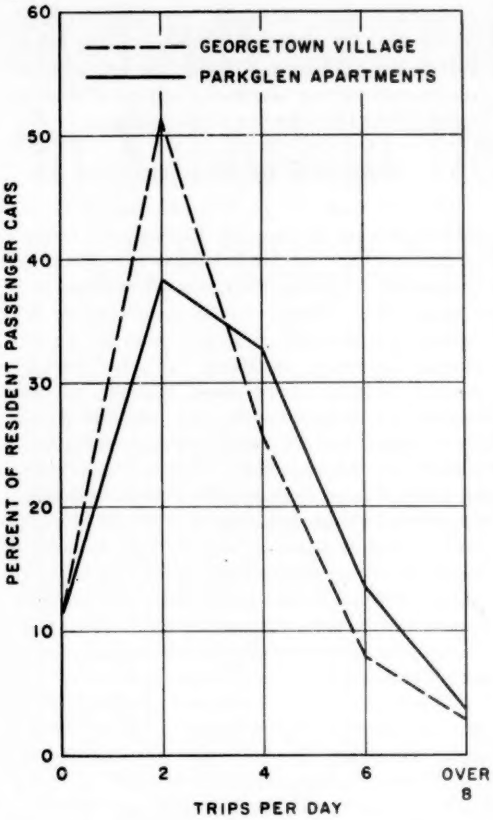

Figure 2.—Frequency of resident passenger-car trips observed at both residential areas.

rise in population density. Information derived from the 1948 Washington, D. C., origin and destination studies shows that average car ownership may be as low as 0.2 vehicle per dwelling unit in densely populated downtown residential areas and as high as 1.7 in some other residential areas. It is assumed that one of the major factors in such wide variation in automobile ownership in urban areas is the availability of parking facilities contiguous to the residence.

Such developments as the Parkglen apartments almost universally provide adequate parking for tenants, so availability of residential parking space is not considered to be a factor in automobile ownership in either Parkglen or Georgetown Village. However, it should be pointed out that there possibly were some two-car families in the Parkglen apartment area. There were six pairs of automobiles with consecutively numbered license plates; therefore, there may have been more apartment families without automobiles than the ratio of 0.97 automobile per dwelling unit might imply. On the other hand, there was at least one car for every dwelling unit at Georgetown Village, so even though the numerical difference in average car ownership between the two areas is not great, it is nevertheless a real and significant difference. Thirty percent of the tenants' vehicles at Parkglen apartments were registered in a State other than Virginia, whereas only 5 percent were registered "out-of-State" at Georgetown Village. This difference is attributed to the large turnover of transient population in apartments as compared with individual homes where it is assumed that most, if not all, are resident owned.

In Parkglen apartments, the outbound and inbound vehicle trips between the hours of 6 a. m. and 10 p. m. during the 2 days of study totaled 1,979. Of this total, 57.4 percent were resident trips and 33.2 percent were nonresident; the remaining trips, 9.4 percent, were made by service vehicles such as milk, bread, laundry and dry-cleaning trucks, and school buses. Excluding pedestrian trips, there were 2.91 resident trips per dwelling unit per day; 1.68, nonresident; and 0.48, service-truck trips.

In Georgetown Village, there was a total of 1,496 vehicle trips for the 2-day period: 59.1 percent, resident; 32.3 percent, nonresident; and 8.6 percent, service-truck trips. Again omitting pedestrian trips, there were 2.95, resident; 1.61, nonresident; and 0.43, truck trips. A comparison of the total vehicle trips per dwelling unit for the two areas shows remarkably close agreement—5.07 and 4.99 for Parkglen apartments and Georgetown Village, respectively. The 1948 origin-and-destination traffic study showed an average of 5.5 passenger-car trips per dwelling for the Washington, D. C., metropolitan area which agrees reasonably well with the data developed here.

Parkglen apartments had four times more pedestrian trips than Georgetown Village, 1.30 and 0.31 trips per dwelling unit per day. This difference is attributed to the availability of mass transit and rather extensive shopping facilities within walking distance of Parkglen apartments. Georgetown Village is not served directly by transit, and shopping facilities are over 3 miles distant. Approximately 40 percent of the pedestrians at Parkglen apartments were mass transit users.

This study does not reflect the total move-

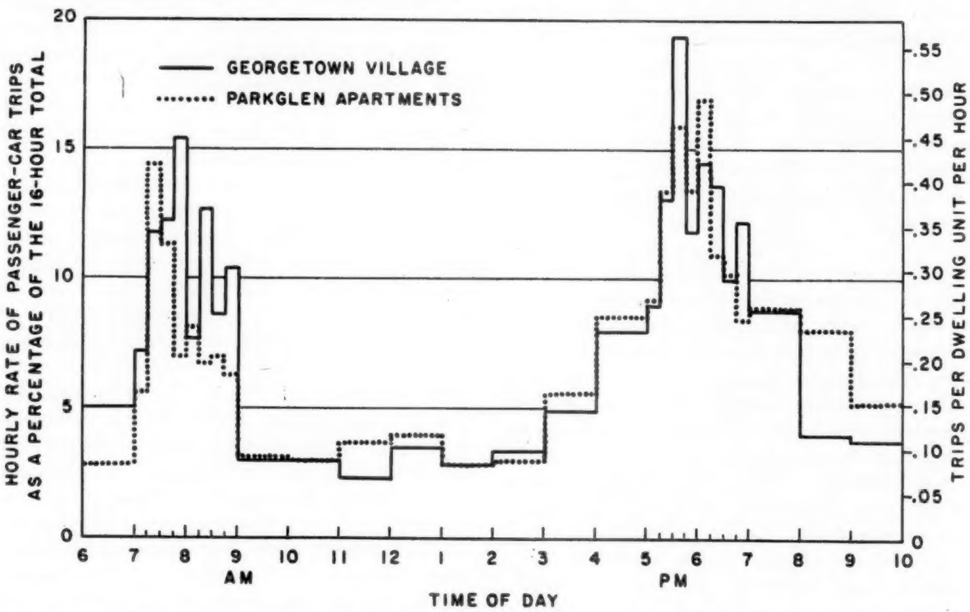

Figure 3.—Hourly rate of resident passenger-car trips with peak hourly rates plotted in 15-minute intervals for both residential areas.

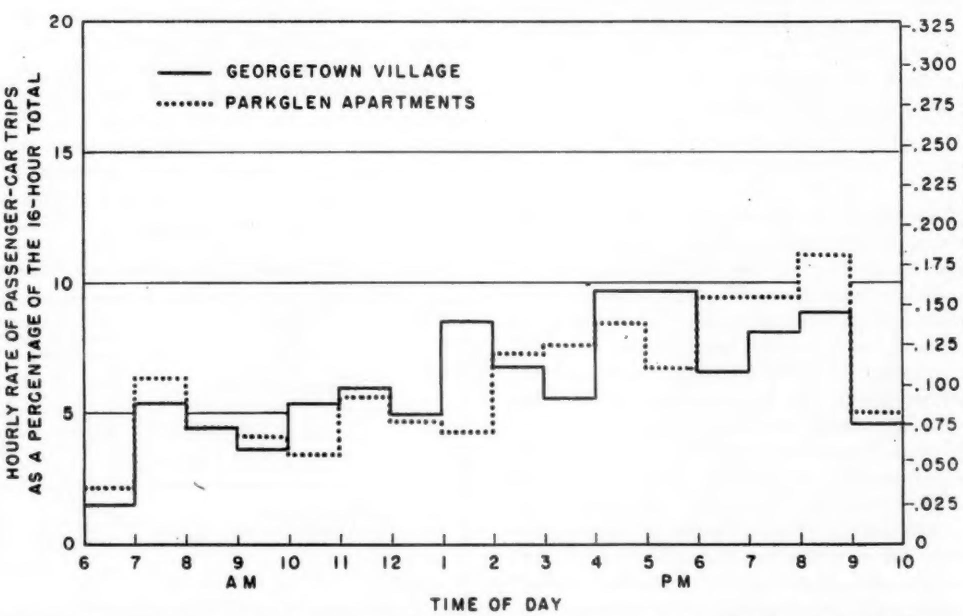

Figure 4.—Hourly rate of non-resident passenger-car trips for both residential areas.

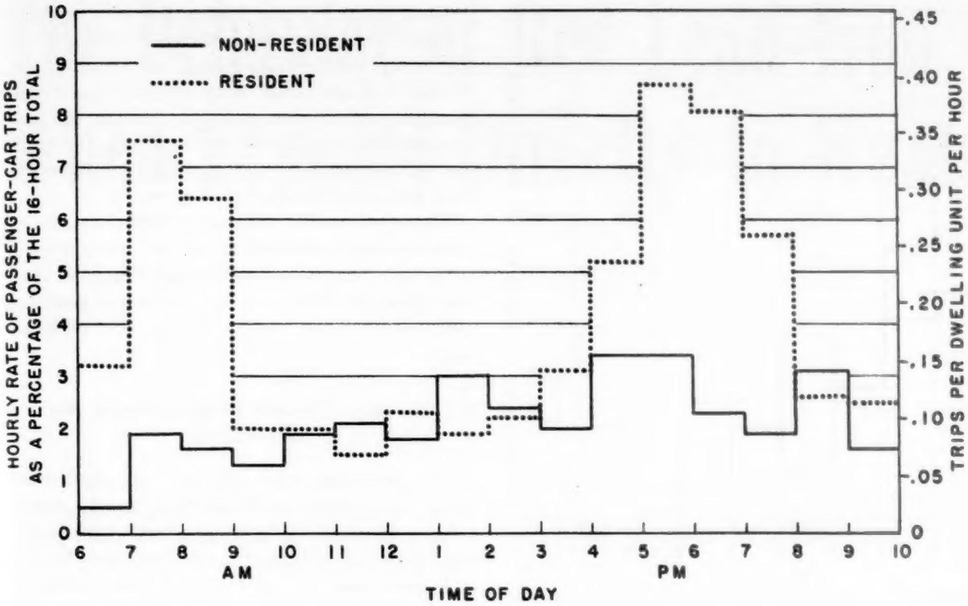

Figure 5.—Hourly rate of resident and non-resident passenger-car trips observed at Georgetown Village.

ments of residents in as much detail as the home-interview origin-and-destination traffic studies. It is believed that trips made by people living in comparatively isolated suburban areas such as Georgetown Village serve more than one purpose, whereas the apartment dwellers who are close to most of their shopping needs make more single-purpose trips. In the home-interview type study, the many purposes of trips are recorded; for example, a journey beginning and ending at home may include stops for a variety of purposes, each of which would be recorded as a separate trip. In the present study, only the movement of traffic to and from the two housing developments was tabulated.

Figure 2 shows the resident passenger-car trip frequencies. In both residential areas, about 12 percent of the resident automobiles made no trips at all outside the area on either day studied. About 51 percent of the automobiles in Georgetown Village, made 2 trips per day, whereas only 38 percent of the resident automobiles at Parkglen apartments made that number. However, a greater number of vehicles at the latter location made three or more trips per day than at the more distant Georgetown Village. This variation is explained by the probability that the more isolated suburbanite at Georgetown Village, though making less frequent trips, actually makes more multipurpose trips than the relatively close-in resident at Parkglen apartments.

As previously mentioned, residents of Georgetown Village owned more cars per dwelling unit than residents of Parkglen apartments and the average number of vehicle trips per day was almost identical for both areas, which indicates that trip frequency does not depend upon automobile ownership in this case. Car occupancy averaged 1.55 persons per car for the Parkglen apartment group, and 1.60 persons per car for Georgetown Village.

Figures 3–8 show the hourly rates of vehicular trips per dwelling unit for the various components of the traffic. It was possible to use the same trips per hour scale to represent both housing developments because, as indicated in table 1, the total trips per day per dwelling unit were almost identical for both areas for each of the components as well as the total vehicular traffic.

Peak Traffic Periods

Figure 3 shows the hourly pattern of resident vehicle trips for both study areas and the peak hourly rates plotted in 15-minute intervals. The morning peak-traffic volumes between 7 and 9 a. m. constitute about 20 percent of the total resident traffic for both study areas. The evening peak period begins to develop about 3 p. m. and does not subside until 8 or 9 p. m., and constitutes approximately 50 percent of the total resident traffic. If it is assumed that all of the morning peak flow (7 to 9 a. m.) is "to work" traffic, an equivalent 20 percent "from work" deducted from the evening peak period leaves 30 percent to be accounted for by residents making additional trips for shopping, recreation, and other purposes.

The California Division of Highways has published a report [2] of a 1947–48 origin-and-destination traffic survey for the city of Sacramento in which is presented a chart (p. 26) of automobile trips similar to figure 3. The morning peak traffic load is about 17 percent of the total and occurs between 7 and 9 a. m. The off-peak volumes are about 5 percent per hour and the evening peak volume, occurring between 3 and 8 p. m., is about 45 percent of the total. The study covered a period of 6 months and involved 328,000 trips.

Figure 3 shows that the elapsed time interval between the Georgetown Village morning and evening peak 15-minute periods is less by about 1 hour than that for Parkglen apartments, 9¾ and 10¾ hours, respectively. Apparently the average resident of Georgetown Village spends less time away from home during the working day than residents of Parkglen apartments which is only half the distance, 6 miles, from the central business district. It is probable that a number of Georgetown Village residents are employed near their homes, thus accounting for the shorter travel period. The National Institutes of Health and the Naval Medical Center, employing several thousand workers, are located about 2 miles from Georgetown Village. These Federal agencies are identified in figure 1.

The nonresident passenger-car traffic pattern is shown in figure 4. Fifteen-minute intervals of traffic movements during the morning and evening peak volumes were not

[2] *Traffic Survey of the Sacramento Area.* State of California, Department of Public Works, Division of Highways, District III. 1947–48.

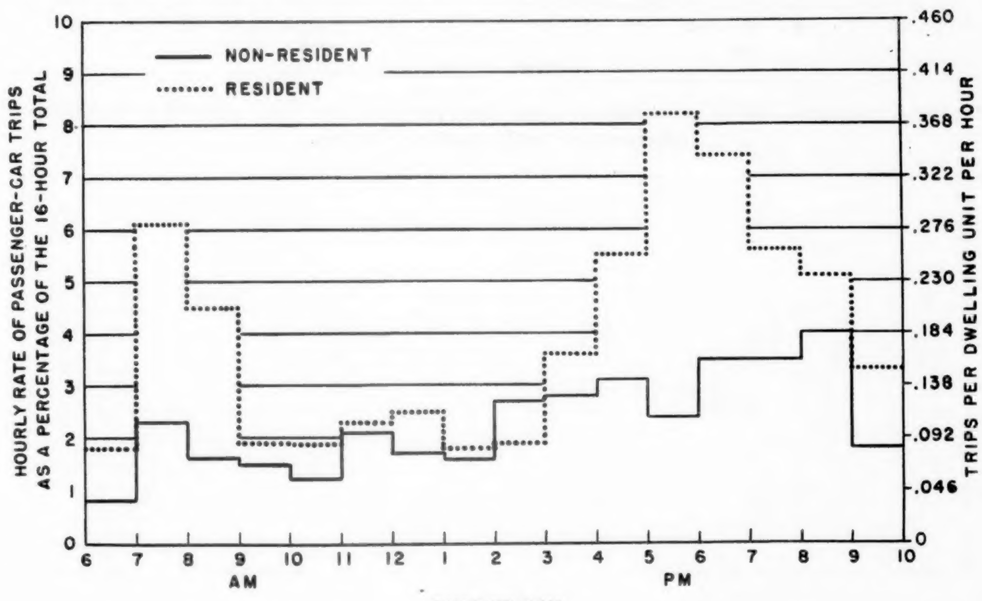

Figure 6.—Hourly rate of resident and non-resident passenger-car trips observed at Parkglen apartments.

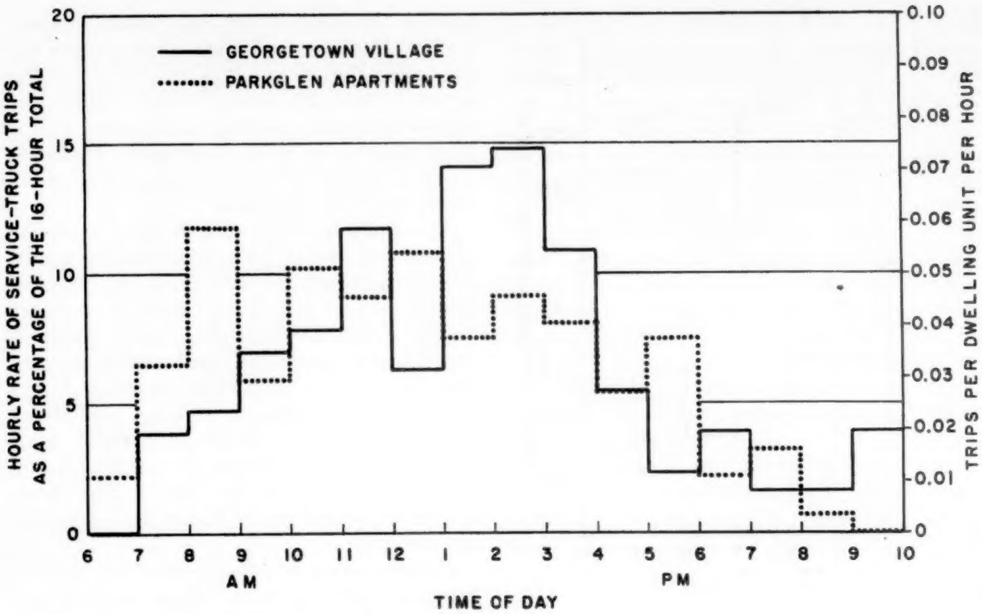

Figure 7.—Hourly rate of service-truck trips for both residential areas.

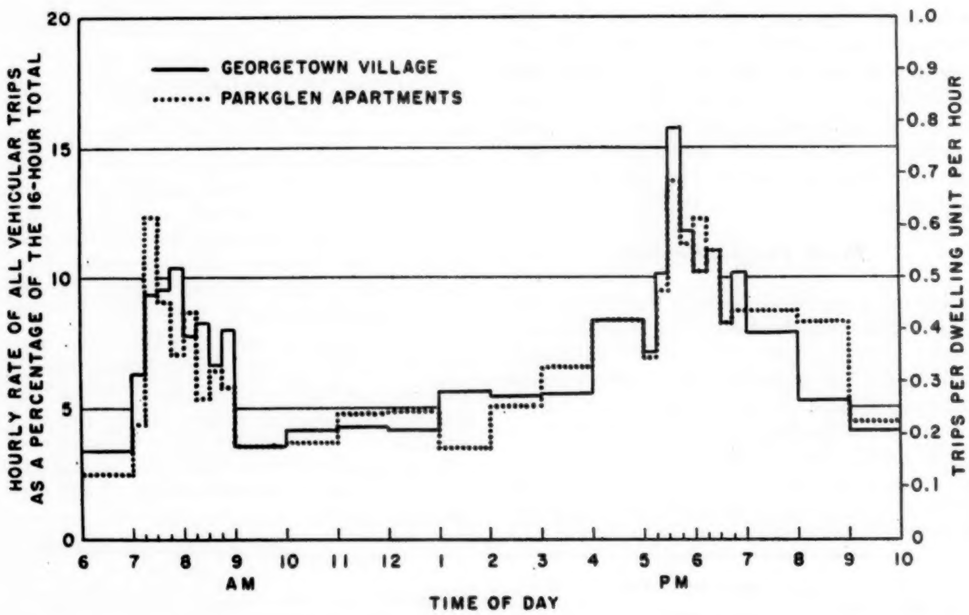

Figure 8.—Hourly rate of all vehicular trips with peak hourly rates plotted in 15-minute intervals for both residential areas.

plotted because little fluctuation was evident. The initial traffic load between 7 and 8 a. m. for both residential areas, particularly at Parkglen apartments, may be due to nonresident carpool drivers arriving to pick up passengers. The volumes then rise slowly throughout the day to a peak of about 0.18 vehicle per hour per dwelling unit at 8 p. m.

Figures 5 and 6 compare resident and nonresident traffic for each of the two areas during each hour of the day. It is significant to note that there is little variation between resident and nonresident traffic during the off-peak hours.

Service-Truck Trips Create No Problem

Service-truck trips to the two residential areas vary considerably in their hourly distribution, as shown in figure 7. Such trips to the apartment development are heaviest in the forenoon and, in general, decline steadily throughout the afternoon. Truck trips to Georgetown Village, however, reached their peak between 1 and 3 o'clock in the afternoon. This dissimilarity may be attributed to the distances involved with respect to the location of the central business district, suburban businesses, and shopping centers. It seems that delivery trucks serve the closer-in areas such as Parkglen earlier in the day. The peak volume of deliveries at Georgetown Village is greater and concentrated in a shorter period of time. The volumes are not particularly significant since peak service-truck travel does not occur at critical traffic flow periods.

All vehicular traffic is combined in figure 8 showing the trip volumes by hour periods and by 15-minute intervals during the hours of peak flow. The initial peak occurring between 7 and 9 a. m. accounts for about 15 percent of the total traffic. From a low point between 9 and 10 a. m., there is a gradual increase until the evening peak between 5 and 6 p. m. This upsurge in the afternoon traffic begins to develop about 2 p. m. and does not subside until after 9 p. m. This period accounts for about 60 percent of all trips. The greatest hourly variation between the two residential areas is about 3 percent but, in general, the traffic patterns of the two areas are quite similar.

The Behavior of Red Lead-Iron Oxide Primers When Exposed Directly to Weathering

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported by WOODROW J. HALSTEAD, Chemist

For special applications where considerable handling of the steel and some delay in applying the field coats of paint is anticipated, the Bureau of Public Roads has obtained good service with a red lead-iron oxide type shop coat primer. This paint is based on a pigment formulation of red lead, iron oxide, and siliceous matter, as in the AASHO Specification M 72, Type III, and a vehicle containing a ratio of three parts of linseed oil to one part of alkyd resin solids together with the necessary thinners and driers.

Direct exposure tests up to 6 years indicate that primers containing this type pigment and ratios of linseed oil to alkyd resin solids from 1:1 to 3:1 all had excellent weather resistance. Primers with a ratio of 5:1 showed some failure at 6 years. Unprotected straight red lead-linseed oil panels failed at 3 years.

DURING the reconstruction of the Philippine Island bridges after World War II, a program carried out under the direction of the Bureau of Public Roads, some difficulties with the shop coat of paint on the structural steel were experienced. Since the steel for a number of these bridges was fabricated in this country, shipment of the material to the job site required considerable handling. As a result, the typically soft film of the straight red lead-linseed oil primer conforming to the American Association of State Highway Officials (AASHO) Specification M 72, Type I, was often mechanically damaged to the extent that proper protection to the steel was not provided.

After a study of the problem, it was recommended that the shop coat be changed to a red lead-iron oxide type similar to that specified in AASHO M 72, Type III, except that the vehicle should contain a ratio of three parts of raw linseed oil to one part of alkyd resin solids instead of the 1:1 ratio in the AASHO specification. The full text of the modified specification is shown in the appendix.

This change was based on considerations of the need for the better wetting characteristics supplied by the raw linseed oil and the need for a paint that dried to a harder film than one containing linseed oil as the drying oil in the vehicle. The 3:1 ratio was chosen after an examination of the drying and visual characteristics of films applied with paints containing various ratios of linseed oil to alkyd resin solids ranging from 1:1 to 5:1. Reports from the field indicated that the adoption of the revised specification successfully eliminated the major portion of the damage to the paint film, and no indication of poor adhesion was apparent.

Since steel shipped overseas or to South American countries may be exposed to severe weathering conditions for relatively long periods of time before the shop coat is covered, the durability of such paints when exposed directly to weathering is of interest. Consequently, a limited investigation of this factor was begun at the same time that the red lead-iron oxide type paint was adopted for use on the Philippine Island bridges. This article reports the results of that investigation.

Procedure and Results

Table 1 shows the properties of seven paints used for exposure panels. Six of these were red lead-iron oxide types while the seventh was a red lead-linseed oil type. As shown, the red lead-iron oxide paints had ratios of linseed oil to alkyd resin solids ranging from 1:1 to 5:1. The alkyd resins were of two types, a long oil linseed-modified resin exemplified by Type II of Federal Specification TT-R-266 [1] and a medium oil linseed-soya-modified as exemplified by Type III of that specification.

Each paint was applied to four panels. The first was a panel that had been allowed to rust and then the rust was mechanically cleaned from one-half of the surface. The remaining three panels were clean and free of rust. One of these was coated to give about the thickness that would be obtained with normal brush application. A second was coated with a thicker than usual film and the third was given two coats, the second being applied 24 hours after the first. All of these panels were then placed on an exposure rack outside the laboratory at an an angle of 45 degrees, facing south. The laboratory is located in Arlington, Va., and therefore the climate is typically that of the Middle Atlantic States. The area is essentially rural, and no industrial gases or salt spray are encountered.

Table 2 shows the average wet film thickness of each coating calculated from the actual weight of paint applied, the measured dry film thickness after 6 years exposure, and the rating of each panel at 1, 3, and 6 years based on a 0–10 point system. The assigned ratings between 4 and 8 were made to be as nearly comparable as possible to the

[1] Copies of individual Federal Specifications may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C.

Table 1.—Properties of primers used in exposure tests

Characteristics of paint	Paint identification No.						
	1	2	3	4	5	6	7
Ratio of linseed oil to alkyd resin solids	3:1	3:1	2:1	2:1	5:1	1:1	(1)
Type of resin (TT-R-266) [2]	III	II	III	II	III	III	-----
Weight per gallon ... pounds	17.3	17.4	17.5	17.6	18.6	17.4	25.4
Pigment ... percent	67.3	67.5	67.8	67.9	71.2	67.6	77.9
Vehicle ... do	32.7	32.5	32.2	32.1	28.8	32.3	22.1
Drying time:							
Set to touch ... hours	3½	4	3½	3½	7	1¼	-----
Dry through within ... do	24	24	24	24	24	16	24
Analysis of pigment:							
True red lead ... percent	61.2	61.7	60.6	61.1	62.5	61.7	98.4
Iron oxide (Fe_2O_3) ... do	12.5	12.8	12.4	12.7	14.0	12.0	0
Siliceous matter ... do	18.7	18.7	18.8	18.8	17.6	17.8	0
Analysis of vehicle:							
Nonvolatile ... do	66.2	68.1	66.3	66.9	69.8	57.7	87.4
Phthalic anhydride [3] ... do	7.3	5.2	9.1	7.1	3.6	15.2	0
Oil acids [3] ... do	78.0	84.4	74.9	77.9	79.2	71.1	-----
Iodine number of fatty acids	-----	-----	-----	-----	-----	-----	184

[1] All linseed oil.
[2] Federal Specification TT-R-266, Type II, is a long oil linseed-modified resin; Type III is a medium oil linseed-soya modified.
[3] On basis of nonvolatile vehicle.

Table 2.—Data for application and durability of exposure panels

Panel identification No.	Surface application conditions	Paint identification No.	Film thickness: Wet [1]	Film thickness: Dry [2]	Ratings [3] after exposure for—: 1 year	3 years	6 years
			Mil.	*Mil.*			
S-1	Half clean-half rust	1	3.5	1.2	10	10	9
1	Clean-normal	1	2.3	1.0	10	10	10
2	Clean-thick	1	4.5	2.0	10	10	10
3	Clean-two coats	1	4.9	2.5	10	10	10
S-2	Half clean-half rust	2	3.1	2.0	10	10	7
4	Clean-normal	2	2.0	.8	10	10	8
5	Clean-thick	2	3.8	1.2	10	10	10
6	Clean-two coats	2	4.8	2.5	10	10	10
S-3	Half clean-half rust	3	3.6	2.0	10	10	8
7	Clean-normal	3	2.5	1.3	10	10	10
8	Clean-thick	3	5.2	2.0	10	10	10
9	Clean-two coats	3	5.4	2.5	10	10	10
S-4	Half clean-half rust	4	3.3	1.5	10	10	8
10	Clean-normal	4	2.5	1.0	10	10	10
11	Clean-thick	4	4.9	2.0	10	10	10
12	Clean-two coats	4	5.3	3.0	10	10	10
S-5	Half clean-half rust	5	3.5	1.5	10	9	6
13	Clean-normal	5	2.5	1.0	10	9	5
14	Clean-thick	5	3.6	1.0	10	9	6
15	Clean-two coats	5	4.7	2.0	10	10	9
S-6	Half clean-half rust	6	4.8	2.0	10	9	7
16	Clean-normal	6	3.4	1.5	10	10	10
17	Clean-thick	6	6.1	3.5	10	10	10
18	Clean-two coats	6	7.6	5.0	10	10	10
RL-1	Clean-normal	7	------	------	10	4	0
RL-2	Clean-normal	7	------	------	10	4	0

[1] Calculated from actual weight of paint applied.
[2] Measured after 6 years.
[3] Ten indicates no deterioration; zero indicates complete removal of film.

ratings that would be assigned under the "Standard Method of Evaluating Degree of Resistance to Rusting Obtained with Paint on Iron or Steel Surfaces," American Society for Testing Materials, Designation D 610–43. Expressed in a general way, it may be considered that panels rated 8 or above are in satisfactory condition and further service would be expected. Panels rated 7 show some signs of deterioration but only a very small amount of scraping and spotting would be necessary before recoating. A rating of 6 or 5 indicates that the paint is no longer adequately protecting the metal. Considerable scraping to remove rust would be necessary before recoating. A rating of 4 or lower indicates complete failure. Complete repriming after removal of rust and old paint would be required.

Discussion of Results

In any consideration of the results obtained from this series of exposure panels, the limited objective of observing the weathering properties of the primers themselves must be kept in mind. The results do not apply to complete paint systems in which the primer is adequately protected by intermediate and top coats. Although the unprotected red lead-linseed oil films were not greatly resistant to the deterioration from weathering forces, it should be noted that the chemical protection provided by this primer was such that no extensive rusting of the steel occurred until the red lead was completely removed from the surface. At 3 years, for example, most of the paint had been removed but in those areas where only a small amount of red lead remained, no rusting had occurred.

The most significant result of this investigation is the excellent weathering resistance of the red lead-iron oxide type primers containing ratios of linseed oil to alkyd resin solids not greater than 3:1. In all cases, primers containing 1:1, 2:1, and 3:1 ratios with either Type II or Type III resins have given excellent protection. No failure is apparent after 6 years except for some indication that films applied over rust—a practice that is definitely not acceptable in field applications—had become sufficiently permeable to moisture that rusting beneath the film had increased. The paint containing a 5:1 ratio of linseed oil to alkyd resin solids began to show some deterioration at about 3 years. At 6 years all of the one-coat applications had failed, although the two-coat application still retained a rating of 9.

The inhibitive action of these primers appeared to be adequate. For films applied over clean metal, there was no indication of failure at the very edge of the panels even though the edges and under surfaces were not coated and showed considerable rusting.

These experiments did not show any failure in adhesion over tight rust. However, since no mill scale or flaked rust was present on these panels, the apparently good adhesion obtained cannot be considered as being indicative of satisfactory adhesion to structural steel for all the combinations tested.

The excellence of straight red lead-linseed oil primers is established by years of satisfactory service, and therefore the superior weathering resistance and harder films obtained by the use of red lead-iron oxide primers may be of little consequence in many applications. For special cases where early and multiple handlings of the steel are necessary or where exposure to severe weathering conditions for a considerable period of time is likely, the red lead-iron oxide primers containing alkyd resin as part of the vehicle are recommended as being more satisfactory for use as shop coats than straight red lead-linseed oil type primers.

Although direct laboratory evidence was not obtained in this investigation, the general behavior of the paints examined and the reports obtained from field applications indicate that a ratio of one part alkyd resin solids to three parts of raw linseed oil is sufficient to provide a film of the desired hardening and drying characteristics without greatly affecting its wetting properties. In this respect, a formulation such as provided for in the Bureau of Public Roads specification shown in the appendix is considered more desirable than paint made as specified in AASHO Specification M 72, Type III.

Appendix

Specifications for red lead-iron oxide paint, used by the Bureau of Public Roads, are as follows:

(*1*) *Pigment.*—The pigment shall be as specified in table I. The iron oxide pigment shall be on a siliceous base (not calcium sulfate). The extracted pigment on analysis shall conform to the quantitative requirements given in table I under the subhead "Extracted pigment."

(*2*) *Vehicle.*—The vehicle shall consist of raw linseed oil blended with a glyceryl phthalate type varnish composed of a linseed oil modified resin together with the necessary driers and volatile thinners. The requirements for the vehicle are also shown in table I. The raw linseed oil shall conform to the Federal Specification TT-O-369. The alkyd resin shall conform to Federal Specification TT-R-266, Type 3. All vehicles shall be free from rosin and rosin derivatives. (The test for rosin subsequently shown under paragraph 11 shall be negative.) The vehicles may contain additional agents such as antioxidents and wetting aids.

(*3*) *Paint quantitative requirements.*—The paint shall meet the quantitative requirements shown in table II.

(*4*) *Condition in container.*—The paint shall not show excessive settling in a freshly opened full can, and shall easily be redispersed with a paddle to a smooth homogeneous state. The paint shall show no curdling, livering, caking, or color separation and shall be free from lumps and skins.

(*5*) *Dilution stability.*—The paint shall remain stable and uniform after reduction of eight parts by volume of the packaged material with one part by volume of mineral spirits conforming to Federal Specification TT-T-291, grade 1.

(*6*) *Brushing properties.*—The paint as received shall brush easily, possess good leveling properties, and show no running or sagging tendencies when applied at a spreading rate

of 500 square feet per gallon to smooth steel vertical surfaces.

(7) *Skinning.*—The paint shall not skin within 48 hours in a three-quarters filled closed container.

(8) *Appearance.*—The paint shall dry to a smooth uniform finish free from roughness, grit, unevenness and other surface imperfections. The paint shall show no streaking or separation when flowed on clean glass.

(9) *Sampling.*—Sampling shall be performed in accordance with Method 102.1 of Federal Specification TT–P–141.

(10) *Storage stability.*—The paint shall show no thickening, curdling, gelling, or hard caking when stored for 6 months from date of delivery in a full, tightly covered container at a temperature of 21 to 32 degrees centigrade (70–90° F.).

(11) *Testing.*—The paint shall be tested in accordance with applicable methods of Federal Specification TT–P–141 and as hereinafter specified. The following tests shall be conducted in accordance with that specification:

Test	*Method*
Percentage of pigment	402.1
True red lead (Pb_3O_4)	707.1
Iron oxide (Fe_2O_3)	714.1
Isolation of vehicle	403.2
Nonvolatile in vehicle (see note a)	405.3
Phthalic anhydride (procedure A)	702.1
Oil acids	703.1
Uncombined water	408.1
Consistency (Krebs-Stormer)	428.1
Coarse particles and skins	409.1
Weight per gallon	401.1
Set to touch time	406.1
Condition in container	301.1
Brushing properties	205.1, 432.1
Spraying properties	204.1, 433.1
Skinning	414.1
Rosin and rosin derivatives (see note b)	503.1
Flexibility (see note c)	622.1
Flash point	429.3
Storage stability	414.2

(*a*) *Method 405.3.*—A gravity convection oven may be used to determine the nonvolatile content of the supercentrifuged vehicle if the procedure outlined in Method 404.1 is modified as follows: Weigh accurately from 0.8 to 1.2 grams of sample (by difference), heat for 1 hour, cool, and weigh. Use the lower value to calculate the percentage of nonvolatile matter.

Table I.—Pigment and vehicle requirements

Ingredients	Requirements, percent by weight	
	Minimum	Maximum
PIGMENT		
Red lead (TT–R–191 Type I, grade C)	65.0	--------
Aluminum stearate	.3	0.4
Red iron oxide pigment (85 percent Fe_2O_3)	15.0	--------
Magnesium silicate	--------	14.7
Mica, 325 mesh	4.0	6.0
EXTRACTED PIGMENT		
True red lead, Pb_3O_4	62.5	--------
Iron oxide, Fe_2O_3	12.5	--------
Siliceous matter	--------	22.0
VEHICLE		
Raw linseed oil	49	--------
Alkyd resin solids	16	--------
Volatile thinner and drier	--------	35

(*b*) *Method 503.1.*—Make the test on a portion of the isolated vehicle.

(*c*) *Method 622.1.*—Apply the paint to flat tin panels with a 0.002 inch (wet film thickness) doctor blade or by other suitable means. The baking and bending constant shall be as follows:

Allow the paint to dry for 18 hours at room temperature. Bake for 5 hours at 75° plus or minus 2° C. (167°±4° F.). Condition the panel for a minimum of 15 minutes at 25° plus or minus 0.1° C. (77°±0.2° F.), and then bend at that temperature over ¼-inch mandrel. Examine for cracks in a strong light at 7 diameter magnification.

(12) *Dry through time.*—Prepare a panel as in paragraph 1 of Method 406.1. The film shall be considered dry through when it cannot be distorted or removed by the following test:

Place the panel in a horizontal position at a height such that when the thumb is placed on the film the arm of the operator is in a straight line from wrist to shoulder. Bear downward on the film with the full area of the ball of the thumb, exerting the maximum pressure of the arm only (not body weight), and simultaneously turn the thumb through an angle of 90 degrees in the plane of the film. Examine for loosening, detachment, wrinkling, or other evidence of distortion of the film.

Table II.—Quantitative requirements of paint

Characteristics	Quantitative requirements	
	Minimum	Maximum
Pigment, percent by weight	68	--------
Nonvolatile vehicle, percent by weight of vehicle	65	--------
Phthalic anhydride, percent by weight of nonvolatile vehicle	7	--------
Oil acids, percent by weight of nonvolatile vehicle	78	--------
Uncombined water, percent by weight of paint	--------	0.5
Coarse particles and skins (retained on No. 325 sieve), percent by weight of pigment	--------	2.0
Consistency (Krebs-Stormer), shearing rate at 200 r. p. m.:		
Grams	155	225
Equivalent KU	73	86
Weight per gallon...pounds	17.0	--------
Drying time:		
Set to touch.....hours	--------	7
Dry through.....hours	--------	24
Flash point(degrees, Fahrenheit)	86	--------

(13) *Appearance of paint coat.*—Examine the panels prepared for brushing and spraying properties. Flow a portion of the paint on a clean glass plate. Let dry in a nearly vertical position at room temperature and examine 4 inches from the top.

(14) *Analysis of pigment (siliceous extender and mica).*—Transfer 0.5 grams of the pigment to a 400 ml. beaker. Add 15 ml. of concentrated hydrochloric acid and heat gently until the iron oxide and red lead are completely dissolved. Evaporate to dryness and bake at 105–110 degrees centigrade for 1 hour. Moisten the residue with a few drops of concentrated hydrochloric acid, dilute to 100 ml. with hot water, boil, filter, and wash with hot water. Transfer the paper and contents to a weighed porcelain crucible; ignite, cool, and weigh; and examine the residue microscopically to determine the presence of mica. Calculate the percentage and report this residue as the siliceous extender and mica.

Driver Behavior

(*Continued from page 205*)

centage of vehicles traveling below 40 miles per hour was lowest when the signs and the edge stripes were present. In general, it appears that edge stripes of the type studied in Oregon reduced vehicle speeds more than the special signs. Signs in combination with the edge stripes did not seem to influence vehicle speeds.

The primary objective for studying edge stripes and signs in Oregon was to determine what markings are effective in reducing the use of shoulders by commercial vehicles. In table 10, the average lateral positions, the percentages of vehicles encroaching on the shoulder, and the average clearances between the bodies of meeting vehicles are given for the five conditions of study. It appears that commercial vehicles used the shoulders less when only the stripes were present than when the stripes were supplemented with signs. During the daytime, more than 40 percent of the trucks encroached on the shoulder during normal operating conditions. When the stripes were present the encroachment was reduced to 13 percent and adequate clearances between the bodies of trucks meeting other vehicles were still maintained. Only 6 percent of the trucks encroached on the shoulders at night when edge stripes were used.

A 4-inch-wide solid yellow reflectorized stripe placed 13 feet from the center of a two-lane, 24-foot bituminous pavement or 1 foot on the paved shoulder was very effective in reducing the encroachment on shoulders, especially by trucks. Such edge stripes also reduced vehicle speeds about 3 miles per hour. Signs on the extreme edge of the shoulder with the legend NO TRAVELING ON PAVED SHOULDERS had only a minor effect on shoulder usage.

Trends of Factors Used in Determining the 30th Highest Hourly Traffic Volumes

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM P. WALKER, Highway Transport Research Engineer

Within the past two decades, highway officials have adopted the policy of designing highways to meet the traffic load of the 30th highest hourly volume of the year for which the facility was being built.

The present analysis of automatic traffic recorder data for rural highways reveals that the 30th-hour factor exhibits a tendency to decline slightly with the passing of time, rather than to remain stable as past studies have indicated. Records for 160 traffic recorder stations in continuous operation from 1946 through 1953 provided the basic data for the analysis. All classes of rural highways were represented and the coverage included 26 States.

The average factor for these stations declined at the average rate of 0.11 per year over the period 1946–53, but a wide variation in the rate of decline was found among different stations. Generally speaking, roads with volumes of more than 3,000 vehicles per day experienced a more rapid rate of decline in the factor than the roads with lesser traffic volumes. Also, 30th-hour factors of 15 or greater experienced a more rapid rate of decline than factors smaller than 15. Tabular data relating the annual change in the 30th-hour factor to both the magnitude of the factor and the average daily traffic volumes are included in this article.

IN 1940 an investigation was made of the relation between traffic volumes during peak hours and the annual average daily traffic volumes on a number of rural highways. Records from automatic traffic recorders provided the basic material for this investigation. Comparatively few of these recorders were in use at that time and none had been in continuous service for longer than 3 or 4 years. Nevertheless, a striking correlation was found between peak-hour volumes and the average daily traffic on rural highways. The authors of a report [2] on these investigations recommended that highways be designed to accommodate a volume of traffic at least as great as that which would occur during the 50th highest hour of the year, but no greater than that for the 30th highest hour. The American Association of State Highway Officials adopted the policy that highways should be designed for the 30th highest hourly volume of the year for which the highway was being built.

The work was reviewed in 1945 when better counter coverage had been established and the period of continuous operation had been extended. The results of this review [3] strengthened the recommendations of the 1940 study, and drew the additional conclusion that for any particular facility the ratio of the 30th highest hourly volume to the average annual daily volume changed very little, if at all, from year to year. This ratio, normally expressed as a percentage figure, is often referred to as the 30th-hour factor. It is a coefficient which, if known for a particular road, will yield the 30th highest hourly volume when applied to the average annual daily traffic on that road.

Purpose of Study

The present reexamination of traffic data is for the purpose of detecting any trends that may exist in the magnitude of the 30th-hour factor. If the factor for any road is indeed fixed or stable, as indications in the past have suggested, then a means is assured for estimating design-hour volumes with a degree of confidence as great as that for the estimate of the average daily traffic. If there is any tendency for the factor to become either larger or smaller with the passing of time, then the rate of change should be determined so that appropriate adjustment can be made in the design-hour volume for any future year. Unless proper adjustment of the factor is made, facilities designed for future traffic will be either overdesigned for their traffic load or they will become congested in a shorter period of time than anticipated, even though the future daily traffic is accurately predicted.

Records for 160 counters that were in continuous operation during the period 1946–53 are used in this analysis. All of these counters were located on rural highways of the 26 States shown in table 1. All classes

Table 1.—Stations included in the analysis of trends in 30th-hour factors

State	Number of stations	Numerical code assigned to stations [1]
Connecticut	18	1, 3-6, 8-11, 13, 15, 16, 18, 20-22, 25, 27
Delaware	6	1A, 2B, 3C (N and S), 4D, 5E, 7G
Georgia	4	1, 3, 4, 12
Idaho	2	3, 7
Illinois	3	2, 7, 8
Indiana	6	34A, 42A, 47A, 59A, 72A, 73A
Iowa	4	601, 604, 614, 616
Maine	5	2-4, 7, 9
Maryland	16	1-15, 18
Massachusetts	5	1, 2, 10, 12, 13
Michigan	5	600, 603, 606, 607, 617
Mississippi	4	3, 8, 10, 4A
Missouri	6	1, 2, 5, 12, 22, 26
Nebraska	5	A2, A4, A5, A8, A12
Nevada	4	101, 107, 109, 110
New Hampshire	5	1, 2, 4, 6, 15A
North Carolina	6	1-4, 22, 24
Ohio	4	25, 28, 31, 38
Oklahoma	16	1, 4-6, 8, 9, 10A, 11-15, 17-20
Pennsylvania	7	4, 5, 10, 17, 21, 22, 31
Tennessee	2	3, 11
Texas	5	1, 4, 5, 8, 17
Utah	7	301, 302, 305, 308, 312, 313, 315
Vermont	3	A12-1, C14-2, PR110
Washington	9	1-3, 6-8, 12, 14, 15
West Virginia	3	1, 3, 4
Total	160	

[1] Individuals or agencies having a bona fide interest may obtain detailed traffic data for any station or group of stations identified in this column by addressing the U. S. Bureau of Public Roads, Division of Highway Transport Research, Washington 25, D. C.

[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

[2] *Applications of automatic traffic recorder data in highway planning*, by L. E. Peabody and O. K. Normann. PUBLIC ROADS, vol. 21, No. 11, January 1941.

[3] *Highway Capacity Manual.* Published by the Bureau of Public Roads, 1950, pp. 129–147.

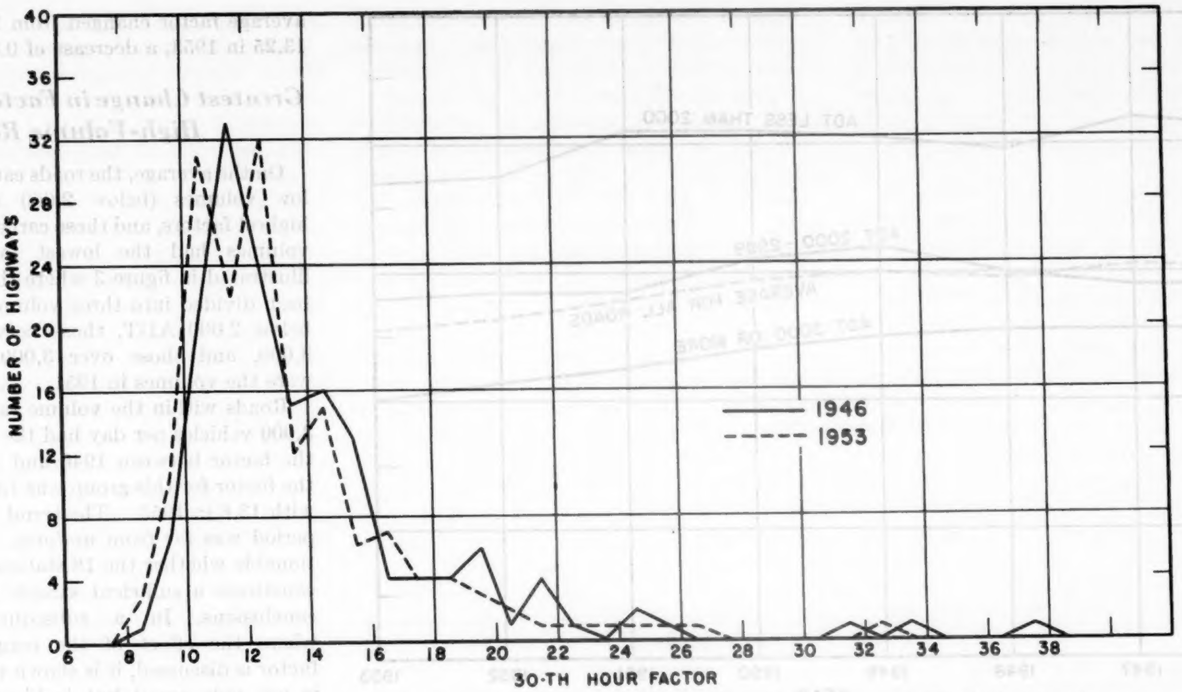

Figure 1.—Distribution of 30th-hour factors for 160 rural highways of all classes in 26 States for the years 1946 and 1953.

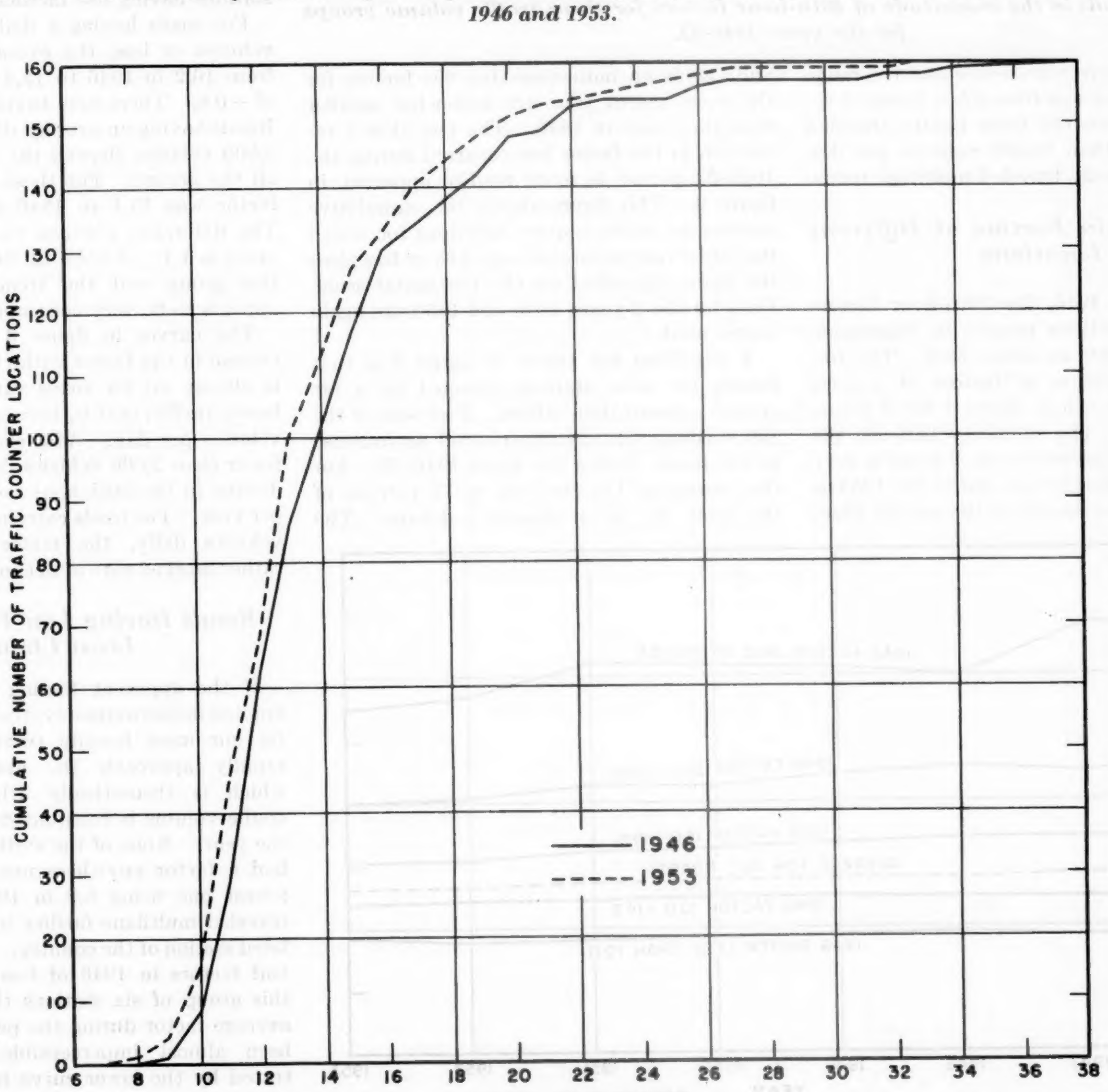

Figure 2.—Cumulative number of traffic counter locations having 30th-hour factors of various values for the years 1946 and 1953.

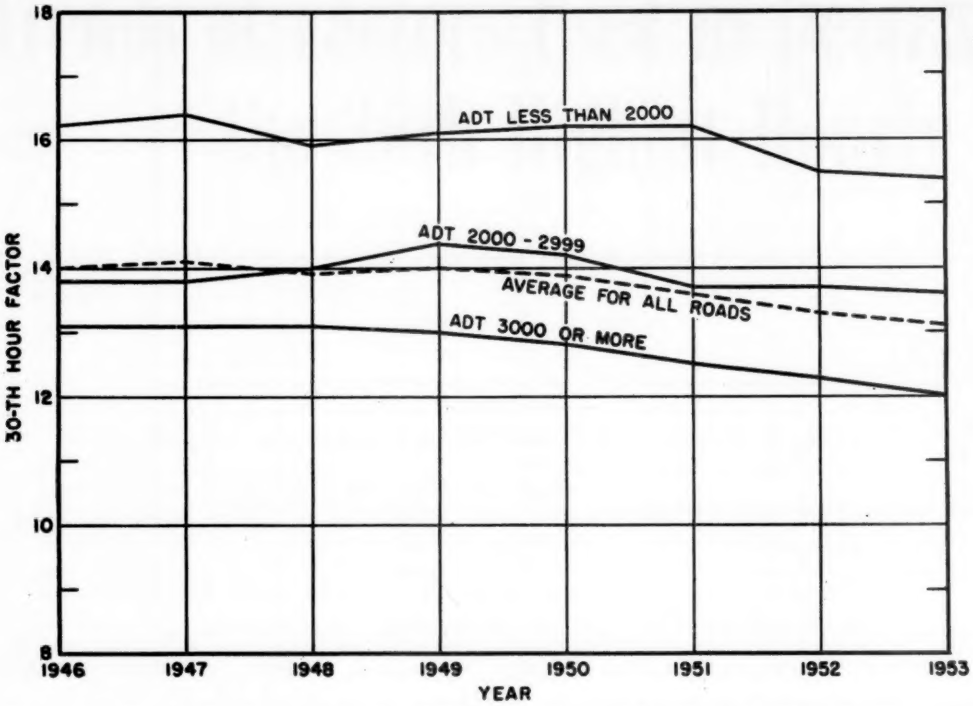

Figure 3.—Trends in the magnitude of 30th-hour factors for three traffic volume groups for the years 1946–53.

of rural roads were represented and the range in traffic volumes was from a few hundred vehicles per day on the most lightly traveled roads to more than 25,000 vehicles per day on the more heavily traveled multilane roads.

Wide Range in Factors at Different Locations

For the year 1946, the 30th-hour factors for the 160 locations ranged in magnitude from less than 9.0 to above 30.0. The frequency of occurrence of factors of various magnitudes is shown in figure 1 for 2 years, 1946 and 1953. The similarity between the distributions of factors for the 2 years is very striking. The distribution curve for 1953 is located slightly to the left of the one for 1946, and this is an indication that the factors for the more recent year are somewhat smaller than they were in 1946. The fact that a reduction in the factor has occurred during the 1946–53 period is more readily apparent in figure 2. This figure shows the cumulative number of traffic counter locations for which the 30th-hour factors are equal to or less than the values specified on the horizontal scale. Data for the 2 years, 1946 and 1953, are again represented.

A condition not shown in figure 2 is that factors for some stations changed by a far greater amount than others. Forty-six of the 160 stations actually experienced an increase in the factor during the years 1946–53. For the remaining 114 stations, or 71 percent of the total, the factor showed a decline. The average factor changed from 14.07 in 1946 to 13.25 in 1953, a decrease of 0.82.

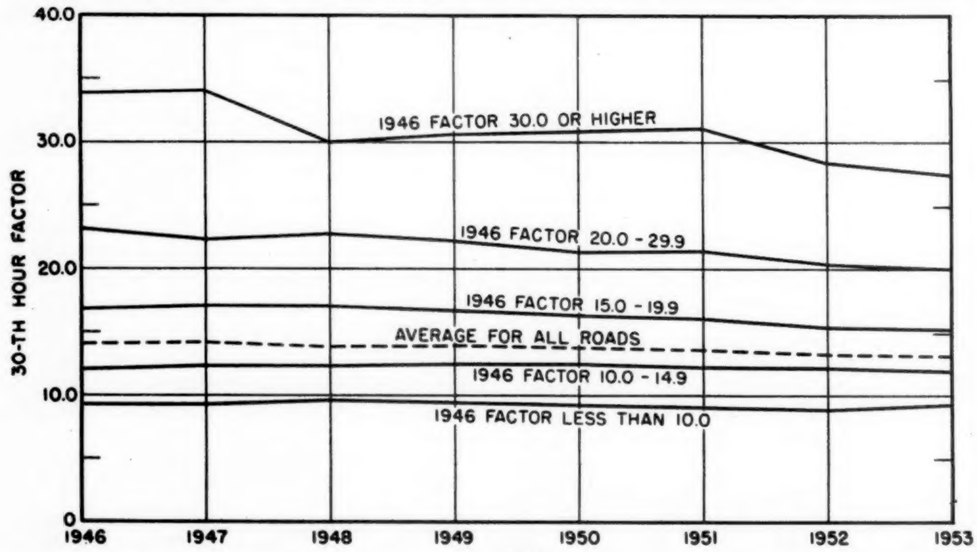

Figure 4.—Trends in the 30th-hour factor for roads having factors of various magnitudes for the years 1946–53.

Greatest Change in Factors Occurs on High-Volume Roads

On the average, the roads carrying relatively low volumes (below 2,000 ADT) had the highest factors, and those carrying the heaviest volumes had the lowest factors. This is illustrated in figure 3 where the stations have been divided into three volume groups: those below 2,000 ADT, those between 2,000 and 3,000, and those over 3,000 ADT. These were the volumes in 1951.

Roads within the volume range of 2,000 to 3,000 vehicles per day had the least change in the factor between 1946 and 1953. In 1946 the factor for this group was 13.8 as compared with 13.6 in 1953. The trend throughout the period was far from uniform, and it is questionable whether the 18 stations in this group constitute a sufficient sample to justify firm conclusions. In a subsequent paragraph where the effect of the magnitude of the factor is discussed, it is shown that the sample is not only small but is biased in favor of stations having low factors.

For roads having a daily volume of 2,000 vehicles or less, the average factor declined from 16.2 in 1946 to 15.4 in 1953, a change of −0.8. There were 43 stations in this group. Roads having an average daily volume of over 3,000 vehicles showed the greatest change of all the groups. For these roads the average factor was 13.1 in 1946 and 12.0 in 1953. The difference between the factors for these years is 1.1. A total of 99 stations comprise this group and the trend over the years appears to be very consistent.

The curves in figure 3 suggest that the change in the factor with the passage of time is almost nil for roads carrying moderately heavy traffic; that is, between 2,000 and 3,000 vehicles per day. As a class, roads carrying fewer than 2,000 vehicles daily have shown a decline in the 30th-hour factor of about 0.11 per year. For roads carrying more than 3,000 vehicles daily, the factor has shown the rather marked rate of decline of 0.16 per year.

Roads Having Low Factors Show Least Change

If the apparent finding just stated were applied indiscriminately, the 30th-hour factors for our most heavily traveled roads would rapidly approach the absolute minimum, which is theoretically 4.15, assuming the traffic volume is constant during all hours of the year. None of the stations in the sample had a factor anywhere near this small—the lowest one being 8.2 in 1946 for a heavily traveled multilane facility in a densely populated section of the country. Only six stations had factors in 1946 of less than 10.0. For this group of six stations the change in the average factor during the period 1946–53 has been almost imperceptible. This is illustrated by the lower curve in figure 4, which shows the trend in the factor for five ranges in the values for the 30th-hour factor.

It is only for the groups of factors having

values of 15.0 or more that any appreciable change in the average factor with passage of time is to be noted. The stations having the highest factors are those showing the greatest change. Average factors for stations having a 1946 factor of 30 or more have declined from 33.7 in 1946 to 27.4 in 1953, a reduction of 6.3. For stations having 1946 factors between 20 and 30, the change in the average factor during the period amounts to 3.0, while the corresponding change for stations having 1946 factors within the range of 15.0–19.9 is 1.8. For factors of less than 15.0, which comprise 72 percent of the total number of stations studied, the change in the factor with passage of time is negligible. This further accounts for the apparent stability of the average factor for roads having volumes between 2,000 and 3,000 ADT, as shown in figure 3, because 14 of the 18 stations within this volume group had factors smaller than 15.0.

The average of the factors for the six stations having 30th-hour factors of less than 10.0 suggests that as a practical matter the irreducible minimum value must be in the neighborhood of 9.5. There are, of course, exceptions to this rule but they are few and limited perhaps to facilities that are so heavily overladen that travel habits are influenced excessively by a high degree of congestion.

Group Factors Decline Although Some Stations Show Increase

The foregoing discussion has shown that if the stations are classified into groups according to either traffic volume or magnitude of factor, the average of the factors for any group shows at least a slight tendency toward decreasing with passage of time. It has also been stated that when the stations are considered individually, almost 30 percent of them experienced an increase in the 30th-hour factor. When the stations are grouped according to magnitude of factor, some of the groups will include stations for which there was an increase in the factor, but it is evident that the effect of these stations is more than offset by the stations within the group that experienced a decrease in the factor.

The groups of stations that showed the least change in the factor—the groups having factors of less than 15—were those that included most of the stations for which there was an increase in the factor. Figure 4 shows that the groups of stations having factors of less than 15 experienced very little change in the average factor. Of the 115 stations for which the factor in 1946 was less than 15.0, 41 stations experienced an increase and 74 experienced a decrease in the factor. Of the 45 stations having factors of 15 or more, only 5 experienced an increase in the factor. Thus, if the factor for any station is less than 15, there is better than one chance in three that the factor will increase rather than decrease with the passage of time. Also, if the factor is greater than 15, there is very little likelihood that it will increase at all.

A somewhat different and more refined interpretation can now be made of figure 3 than was permissible prior to examination of figure 4. It is now apparent that the magnitude of the factor and the traffic volume act in combination to influence the trend in the factor with passage of time. It has been established that factors for roads carrying volumes in excess of 3,000 ADT decline at a more rapid rate than those with lesser volumes and, additionally, that factors of 15 or more will decline at a more rapid rate than will smaller factors. The magnitude of the factor seems to exert a greater influence on the rate of change than does traffic volume. Roads with volumes in excess of 3,000 ADT in combination with factors of 15.0 or more experience the highest rate of decline in the factor. The lowest rate of decline in the factor is experienced by roads having factors of less than 15.0, regardless of the traffic volume. Perhaps there are numerous variables other than traffic volume and magnitude of factor which influence the trend, but the amount of information available for this analysis imposes rather stringent limitations on their detection and evaluation.

With the passing of years the number of stations having high factors within a given sample will diminish and the average rate of decline will be reduced. When the factor for a particular facility reaches a value somewhere between 9.5 and 15.0, depending upon such things as the character of traffic and geographic location, the decline in the factor will be arrested and very little change may be expected thereafter. If the conditions surrounding the minimum values lying within the range of 9.5 to 15.0 could be ascertained, the results of this analysis would be greatly enhanced, but this does not appear to be possible from the data at hand.

Summary

The findings thus far discussed are as follows:

1. The trend in the 30th-hour factor for 160 stations in continuous operation over the 1946–53 period has been a decline of 0.11 per year, on the average, but the rate of decline varied widely among different roads.

2. Roads having volumes greater than 3,000 ADT in combination with factors of 15 or above are those which, as a class, experience the most rapid rate of decline in the factor.

3. The lowest average rate of decline (almost zero) is experienced by roads having 30th-hour factors of less than 15.0, regardless of traffic volume.

4. The fact that there is little change in the average factors for the groups of roads having factors of less than 15.0 is due in part to the phenomenon that within these categories there is almost a one-in-three chance that any given station will experience an increase rather than a decrease in the factor with passage of time.

5. The irreducible minimum value for the 30th-hour factor may be taken as about 9.5 for rural highways. In some geographical areas and under certain conditions yet to be defined, the minimum value of the factor may be as high as 15.0, but a factor lower than 9.5 may be accepted as a definite indication that travel desires are being suppressed.

Application of Findings

As in the case of many investigations of this type the best that can be hoped for is the development of broad indications which, if applied, will provide results that are more nearly correct than would have been possible in the absence of the study. To be of maximum value to the user, the findings of the investigation must be accurate and in considerable detail. When measured against this standard, the study reported here is somewhat deficient because it does not answer fully the question as to why traffic patterns for some roads behave differently from others in the course of time. If applied wisely, however, such facts as have been brought to light should permit a far more accurate estimate of future 30th-hour factors than would otherwise be possible.

As an aid in applying the results of the study, a table of suggested annual changes in the 30th-hour factor for various combinations of factors and traffic volumes has been prepared. These suggested annual changes, as shown in table 2, approximate the average changes found in the statistical analysis. For traffic volumes other than those shown in the column headings of table 2, interpolation between columns is recommended. The suggested method is given in the footnote to table 2 and in the two examples that follow. The volumes of 1,500, 2,500, and 3,500 ADT, shown in the column headings of the table, were selected to aid the user of the data in

Table 2.—Annual change in 30th-hour factor for various combinations of factors and traffic volumes

30th-hour factor	Annual change in 30th-hour factor [1] for ADT of—		
	1,500	2,500	3,500
Pct.	*Pct.*	*Pct.*	*Pct.*
Below 10.0	0.00	0.00	−0.08
10.0–10.9	.00	.00	−.10
11.0–11.9	.00	.00	−.12
12.0–12.9	.00	.00	−.15
13.0–13.9	.00	−.01	−.18
14.0–14.9	−.01	−.02	−.22
15.0–15.9	−.02	−.04	−.27
16.0–16.9	−.03	−.07	−.31
17.0–17.9	−.05	−.10	−.36
18.0–18.9	−.08	−.13	−.41
19.0–19.9	−.10	−.17	−.48
20.0–20.9	−.14	−.20	−.53
21.0–21.9	−.18	−.24	−.59
22.0–22.9	−.21	−.29	−.65
23.0–23.9	−.25	−.34	−.71
24.0–24.9	−.30	−.39	−.79
25.0–25.9	−.35	−.44	−.83
26.0–26.9	−.40	−.50	−.90
27.0–27.9	−.46	−.55	------
28.0–28.9	−.52	−.61	------
29.0–29.9	−.58	−.67	------
30.0–30.9	−.63	−.74	------
31.0–31.9	−.70	−.81	------
32.0–32.9	−.78	−.90	------
33.0–33.9	−.83	------	------
34.0–34.9	−.90	------	------

[1] For volumes of less than 1,500 ADT, use values shown in column 2; for volumes within the range of 1,500 and 2,500 ADT, interpolate between values shown in columns 2 and 3; for volumes within the range of 2,500 and 3,500 ADT, interpolate between values shown in columns 3 and 4; for volumes of 3,500 ADT and over, use values shown in column 4.

interpolating; actually, the average daily volumes for the groups of stations exhibiting annual changes in the 30th-hour factor were 1,200, 2,500, and 6,500, respectively.

Example 1

The present (1956) volume on a rural road is 5,300 ADT. The 30th highest hourly volume in 1956 was 778 vehicles per hour, yielding a 30th-hour factor of 14.30. It is estimated that the ADT in 1970 will be 10,600, or double the present volume. Estimated volumes for the intervening years between 1956 and 1970 are as shown in column 2 of table 3. What will be the 30th-hour factor for 1970?

Solution.—Since the volume for all years is above 3,500, column 4 of table 2 applies to this example. No interpolation is required. The annual change for 1956 is found in column 4 of table 2. The change of −0.22 is applied to the 1956 factor of 14.30 to yield a 1957 factor of 14.08. Thirtieth-hour factors for the succeeding years are obtained in a similar manner. The rate of change diminishes as the factor becomes smaller.

Example 2

The present (1956) volume on a rural road is 1,400 ADT. The 30th-hour volume in 1956 was 259 vehicles per hour, yielding a 30th-hour factor of 18.50. The estimated rate of traffic growth is about 10 percent per year in the following 14 years, as indicated in column 5 of table 3. What will be the 30th-hour factor in 1970?

Solution.—Since the ADT changes from an initial value of less than 1,500 to a final value of over 3,500, columns 2, 3, and 4 of table 2 are used in this example. The factor for 1957 is obtained by subtracting the annual change for 1956 (0.08, column 2, table 2) from the 30th-hour factor for 1956 (18.50). The procedure is repeated for each succeeding year to 1970, with the resulting 30th-hour factor for 1970 being 15.82. During the period 1957–61, the ADT is between 1,500 and 2,500 and the annual change is obtained by interpolating between −0.08 in column 2 of table 2 and −0.13 in column 3. During the period 1963–65, the ADT is between 2,500 and 3,500 and the annual change is determined by interpolating between −0.10 in column 3 and −0.36 in column 4. For the period 1966–69, the volume is greater than 3,500 and no interpolation is required. For these years the annual change is taken directly from column 4 of table 2.

Table 3.—Examples of the application of annual changes in the 30th-hour factor for various combinations of factors and traffic volumes

Year	Example 1			Example 2		
	Average daily traffic	30th-hour factor	Annual change	Average daily traffic	30th-hour factor	Annual change
1956	5,300	14.30	−0.22	1,400	18.50	−0.08
1957	5,560	14.08	−.22	1,550	18.42	−.08
1958	5,850	13.86	−.18	1,700	18.34	−.09
1959	6,150	13.68	−.18	1,875	18.25	−.10
1960	6,460	13.50	−.18	2,050	18.15	−.11
1961	6,790	13.32	−.18	2,250	18.04	−.12
1962	7,130	13.14	−.18	2,500	17.92	−.10
1963	7,500	12.96	−.15	2,750	17.82	−.17
1964	7,880	12.81	−.15	3,000	17.65	−.23
1965	8,280	12.66	−.15	3,300	17.42	−.31
1966	8,700	12.51	−.15	3,600	17.11	−.36
1967	9,140	12.36	−.15	3,950	16.75	−.31
1968	9,600	12.21	−.15	4,400	16.44	−.31
1969	10,100	12.06	−.15	4,800	16.13	−.31
1970	10,600	11.91	--------	5,300	15.82	--------

AE–55 Indicator

(*Continued from page 207*)

indicator gave results averaging about 1 percentage point too high. For air contents of over 6.0 percent the AE–55 indicator gave values averaging 1 percentage point too low. These values indicate that it might be feasible to prepare a correction curve for the AE–55 indicator readings. Figure 4 indicates the amount by which the reading for the AE–55 indicator should be corrected to agree with values for the pressure meter. In this figure the circles indicate the results for each mix, whereas the crosses are the average values given in table 2.

Field Testing of AE–55 Indicator

A limited number of tests were made in the field to determine the accuracy and usability of the apparatus on a paving job. Tests were made on concrete containing 4 percent air as determined by a pressure air meter. The values obtained with the AE–55 indicator were between 3.5 and 4.5 percent with no correction for the mortar content. The corrected values would be from 3.2 to 4.1 percent. If these values were further corrected, as shown by the curve in figure 4, the values obtained would be 3.9 to 4.3 percent which agree closely with the pressure air meter determination. The determinations were made by four different engineers, three of whom had not seen the apparatus before. The device was very favorably received because of its small size and the rapidity with which the test could be made in the field.

Summary

The AE–55 indicator is found to be an apparatus of considerable merit for use in the determination of the approximate air content of concrete in the field, provided the amount of mortar in the concrete is known. The test can be completed in less than 3 minutes and the apparatus can be carried in one's pocket. Attention is called, however, to the small amount of mortar used in a test. To insure the most reliable results, at least three tests should be made for each determination of air content.

The AE–55 indicator method is considered not suitable as a replacement of the pressure or gravimetric methods for the air content of concrete, but is useful as a supplementary test. It does appear to be of most value for use in determining the uniformity of the air content from batch to batch of concrete when no change in the materials or proportions occurs. It may also be used as a rapid check to determine whether the air content is within the specification limits. In no case, however, should the AE–55 indicator method be considered suitable for replacing any of the standard methods mentioned.

U. S. GOVERNMENT PRINTING OFFICE: 1957

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

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- 1941, 15 cents. 1948, 20 cents.
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Public Roads Administration Annual Reports:
- 1943; 1944; 1945; 1946; 1947.
- *(Free from Bureau of Public Roads)*

Annual Reports of the Bureau of Public Roads:
- 1950, 25 cents. 1953 (out of print). 1956, 25 cents.
- 1951, 35 cents. 1954 (out of print).
- 1952, 25 cents. 1955, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
Construction of Private Driveways, No. 272MP (1937). 15 cents.
Criteria for Prestressed Concrete Bridges (1954). 15 cents.
Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.
Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.
Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.
Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.
Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.
First Progress Report of the Highway Cost Allocation Study, House Document No. 106 (1957). 35 cents.
General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.
Highway Bond Calculations (1936). 10 cents.
Highway Bridge Location, No. 1486D (1927). 15 cents.
Highway Capacity Manual (1950). $1.00.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
- 1945 (out of print). 1949, 55 cents. 1953, $1.00.
- 1946, 50 cents. 1950 (out of print). 1954, 75 cents.
- 1947, 45 cents. 1951, 60 cents. 1955, $1.00.
- 1948, 65 cents. 1952, 75 cents.

Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.25.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate,* 15 cents.

PUBLICATIONS (Continued)

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Opportunities in the Bureau of Public Roads for Young Engineers (1955). Out of print.
Parking Guide for Cities (1956). 55 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.
Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-57 (1957). $2.00.
Standard Plans for Highway Bridge Superstructures (1956). $1.75.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24–28.

U. S. DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,435	$36,282	$28,403	255.7	$10,131	$5,844	108.7	$77,858	$48,140	955.4	$124,271	$82,387	1,319.8
Arizona	1,396	24,178	21,594	180.0	9,958	8,502	76.3	20,439	16,834	113.3	54,575	46,930	369.6
Arkansas	17,866	40,278	29,260	533.4	14,149	8,514	125.2	33,635	19,141	464.2	88,062	56,915	1,122.8
California	2,668	34,295	14,143	250.5	42,208	31,795	52.4	404,617	203,334	289.9	481,120	249,272	592.8
Colorado	19,706	16,679	12,281	136.6	13,390	9,649	73.3	40,268	27,360	278.4	70,337	49,290	488.3
Connecticut	33,091	2,771	1,474	4.0	26,368	15,940	18.3	13,358	8,869	26.5	42,497	26,283	48.8
Delaware	16,652	2,350	1,203	14.1	3,921	2,198	28.3	14,331	7,955	51.9	20,602	11,356	94.3
Florida	24,745	24,641	19,407	178.2	18,584	12,752	38.1	38,045	21,696	235.7	81,270	53,855	452.0
Georgia	41,552	54,712	34,015	681.6	8,186	4,399	50.1	89,329	50,583	834.1	152,227	88,997	1,565.8
Idaho	20,007	15,691	12,419	117.3	4,986	4,218	32.8	16,873	11,360	199.8	37,550	27,997	349.9
Illinois	26,378	75,227	51,418	569.9	59,991	47,475	75.8	150,410	102,386	883.1	285,628	201,279	1,528.8
Indiana	84,007	24,501	13,295	134.0	15,903	8,344	95.9	44,270	28,010	168.9	84,674	49,649	398.8
Iowa	3,582	52,024	40,277	472.9	11,830	7,142	183.2	67,095	43,097	1,451.7	130,949	90,516	2,107.8
Kansas	5,733	48,570	39,828	897.4	15,157	9,448	287.8	39,188	23,103	1,185.0	102,915	72,379	2,370.2
Kentucky	46,586	10,002	6,381	85.6	4,563	2,944	11.9	50,670	33,647	317.0	65,235	42,972	414.5
Louisiana	26,115	41,263	26,892	173.1	13,910	8,828	72.9	50,176	26,691	323.2	105,349	62,411	569.2
Maine	23,818	8,999	4,842	83.2	2,929	1,560	16.4	18,966	10,085	118.6	30,894	16,487	218.2
Maryland	3,435	24,282	12,482	82.4	16,384	11,638	18.1	62,351	41,303	203.8	103,017	65,423	304.3
Massachusetts	26,648	40,602	25,550	35.0	39,794	26,830	22.6	64,655	35,208	55.1	145,051	87,588	112.7
Michigan	17,818	71,514	55,535	650.2	35,676	19,335	80.7	112,433	75,942	473.7	219,623	150,812	1,204.6
Minnesota	15,686	16,458	13,410	322.1	15,492	11,341	87.8	86,795	59,391	1,424.9	118,745	84,142	1,834.8
Mississippi	6,396	37,607	26,728	685.7	24,078	19,716	121.9	40,745	23,944	814.6	102,430	70,388	1,622.2
Missouri	27,130	30,956	16,173	1,161.7	17,267	13,683	21.6	118,475	72,646	1,374.1	166,698	102,502	2,557.4
Montana	35,351	10,116	6,839	192.9	6,211	3,698	92.6	47,413	33,940	389.6	63,740	44,477	675.1
Nebraska	36,788	21,989	11,909	470.9	10,732	6,969	114.5	36,599	20,786	1,045.3	69,320	39,664	1,630.7
Nevada	24,126	5,437	4,862	39.9	5,132	4,723	22.9	18,240	15,430	245.4	28,809	25,015	308.2
New Hampshire	5,266	13,565	10,785	28.6	1,503	1,151	1.9	20,577	12,377	82.9	35,645	24,313	113.4
New Jersey	43,499	10,203	5,737	70.1	37,494	26,767	22.3	49,200	32,300	39.8	96,897	64,804	132.2
New Mexico	7,947	5,293	3,776	71.7	8,537	6,800	76.9	33,367	27,479	180.6	47,197	38,055	329.2
New York	68,009	38,793	21,613	105.4	61,203	33,145	114.2	438,132	257,205	555.1	538,128	311,963	774.7
North Carolina	44,168	33,204	21,728	325.2	12,102	7,962	133.9	77,999	43,987	864.4	123,305	73,677	1,323.5
North Dakota	13,454	21,279	15,751	1,230.6	10,747	7,382	258.6	26,999	15,895	1,113.5	59,025	39,028	2,602.7
Ohio	23,957	86,781	56,943	191.4	43,579	31,082	41.4	162,362	105,986	244.3	292,722	194,011	477.1
Oklahoma	21,941	30,781	21,824	409.0	24,892	16,710	139.2	49,289	28,804	521.2	104,962	67,338	1,069.4
Oregon	18,003	8,899	7,207	58.0	4,266	3,516	19.3	44,289	31,922	281.8	57,454	42,645	359.1
Pennsylvania	18,592	118,152	82,627	223.5	62,567	43,105	52.6	202,563	125,028	384.6	383,282	250,760	660.7
Rhode Island	4,344	3,890	2,057	9.6	3,884	3,496	0.4	33,208	22,721	26.5	40,982	28,274	36.5
South Carolina	18,356	38,905	29,173	414.7	2,704	2,099	20.0	37,994	22,072	771.4	79,603	53,344	1,206.1
South Dakota	8,632	31,311	24,317	446.0	4,799	2,683	153.9	27,088	18,105	805.6	63,198	45,105	1,405.5
Tennessee	13,376	65,263	49,416	454.7	6,967	3,484	6.4	82,878	46,791	628.2	155,108	99,691	1,089.3
Texas	86,382	19,329	13,125	379.7	52,150	38,066	169.5	177,528	113,182	1,710.8	249,007	164,373	2,260.0
Utah	15,132	17,274	14,034	143.3	2,673	1,991	13.8	15,709	12,489	191.2	35,656	28,514	348.3
Vermont	10,627	4,505	3,057	24.5	235	115	0.4	20,123	13,522	67.9	24,863	16,694	92.8
Virginia	25,727	47,880	37,132	174.0	16,784	9,478	136.8	43,076	24,228	306.3	107,740	70,838	617.1
Washington	24,513	22,351	16,044	210.9	4,796	3,846	48.4	50,577	33,320	303.0	77,724	53,210	562.3
West Virginia	18,527	43,774	31,859	66.0	4,018	2,049	10.9	31,375	16,177	96.1	79,167	50,085	173.0
Wisconsin	52,444	17,097	9,417	356.0	8,131	4,213	60.5	64,833	38,329	467.1	90,061	51,959	883.6
Wyoming	2,089	9,911	8,170	58.3	7,514	5,969	55.1	39,359	30,582	401.7	56,784	44,721	515.1
Hawaii	5,116	4,987	2,493	14.7				4,340	2,076	8.5	9,327	4,569	23.2
District of Columbia	14,791	6,125	4,886	2.9	10,840	7,925	4.6	11,447	8,624	0.6	28,412	21,435	8.1
Puerto Rico	11,536	4,138	1,558	12.8				18,171	8,614	56.1	22,309	10,172	68.9
Alaska	479	11,099	10,025	104.5	2,813	2,577	354.5	2,136	1,953	108.8	16,048	14,555	567.8
TOTAL	1,182,622	1,486,213	1,035,374	13,994.4	842,128	573,096	3,825.6	3,521,853	2,154,649	24,141.2	5,850,194	3,763,119	41,961.2